

SAFER BRIDGE RAILINGS VOLUME 1 SUMMARY REPORT



U.S. Department
of Transportation

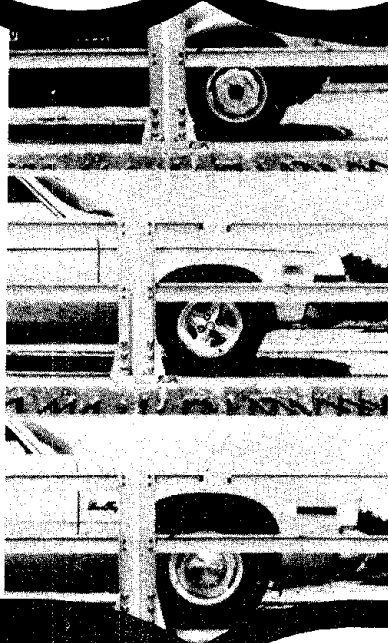
**Federal Highway
Administration**

Research, Development,
and Technology

Turner-Fairbank Highway
Research Center
6300 Georgetown Pike
McLean, Virginia 22101

Report No.
FHWA/RD-82/072

Final Report
June 1984



REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE

U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

FOREWORD

This report presents the results of a study of the performance of selected bridge railing systems intended to meet the current (1976) AASHTO bridge railing specifications. Five in-service railings were selected to provide insight on areas of potential improvement of these specifications. Sixty mph impact tests were conducted with vehicles ranging in size from a 1,800 lb subcompact automobile to a 32,000 lb intercity bus. Tests were also conducted with an instrumented rigid wall to determine the forces that an unyielding barrier must withstand to redirect these vehicles at selected impact angles, assuming no snagging.

Design guidelines were developed to improve the impact performance of bridge railing systems as were suggested improvements to the bridge rail performance standards of NCHRP Report 230. The authors state that the data on which the design guidelines are based is limited and cannot guarantee fully satisfactory impact performance for a given bridge rail design. Thus, while these guidelines should result in improved impact performance of bridge railings, it is suggested that final acceptance of a design be based on performance demonstrated through full-scale crash tests.

This report is one of four volumes titled "Safer Bridge Railings." The other reports are: FHWA/RD-82/073, Volume 2, Appendices A, B, D, and E; FHWA/RD-82/074.1, Volume 3, Appendix C, Part I; and FHWA/RD-82/074.2, Volume 4, Appendix C, Part II. Volume 2 contains elastic and ultimate strength analyses of the tested railings and gives the development of the design guidelines and suggested performance standard revisions. Volumes 3 and 4 contain results of the 30 full-scale crash tests. Direct distribution of Volumes 2, 3, and 4 is being made to the roadside hardware research community.

Volume 1 is being distributed to each regional office, each division office, and to each State highway agency.



Stanley R. Byington
Director, Office of Safety and Traffic
Operations Research and Development
Federal Highway Administration

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1. Report No. FHWA/RD-82/072	2. Government Accession No.	3. Recipient's Catalog No. PB8 6 115698 7AS	
4. Title and Subtitle Safer Bridge Railings Volume 1: Summary Report		5. Report Date June 1984	
		6. Performing Organization Code	
7. Author(s) Eugene Buth With Contributions By A. Arnold, W.L. Campise, T.J. Hirsch, D.L. Ivey and J.S. Noel		8. Performing Organization Report No.	
9. Performing Organization Name and Address Texas Transportation Institute Texas A&M University College Station, Texas 77843		10. Work Unit No. (TRAIS) 31T2-202	
		11. Contract or Grant No. DOT-FH-11-9181	
12. Sponsoring Agency Name and Address Office of Research Federal Highway Administration U.S. Department of Transportation Washington, D.C. 20590		13. Type of Report and Period Covered Final Report September 1976 - May 1983	
		14. Sponsoring Agency Code T-0521	
15. Supplementary Notes FHWA Contract Manager: John G. Viner			
16. Abstract This study consisted of strength analyses of five in-service bridge railing systems, thirty full-scale vehicle crash tests on those railing systems and on a load measuring wall, the development of recommended design guidelines and development of recommended performance standards. The spectrum of vehicles addressed ranged from an 1,800 lb Honda sedan to a 32,000 lb intercity bus. Some of the in-service railings were found to have poor performance in full-scale crash tests. This was primarily due to geometry of the railing; however, deficiencies in strength were also discovered. This report consists of four volumes. Volume 1: Summary Report FHWA/RD-82/072 Volume 2: Appendices A, B, D and E FHWA/RD-82/073 Volume 3: Appendix C, Part I FHWA/RD-82/074.1 Volume 4: Appendix C, Part II FHWA/RD-82/074.2			
17. Key Words Bridge Railing, crash tests, Impact performance, Design guidelines, Performance Standards		18. Distribution Statement No Restrictions. This document is Available through the National Technical Information Service, Springfield, VA 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 165	22. Price \$16.95

METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL WHEN YOU KNOW MULTIPLY BY TO FIND SYMBOL

LENGTH

in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km

AREA

in ²	square inches	6.5	square centimeters	cm ²
ft ²	square feet	0.09	square meters	m ²
yd ²	square yards	0.6	square meters	m ²
mi ²	square miles	2.6	square kilometers	km ²
	acres	0.4	hectares	ha

MASS (weight)

oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons(2000lb)	0.9	tonnes	t

VOLUME

tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft ³	cubic feet	0.03	cubic meters	m ³
yd ³	cubic yards	0.76	cubic meters	m ³

TEMPERATURE (exact)

°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C
----	------------------------	----------------------------	---------------------	----

APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL WHEN YOU KNOW MULTIPLY BY TO FIND SYMBOL

LENGTH

mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi

AREA

cm ²	square centimeters	0.16	square inches	in ²
m ²	square meters	1.2	square yards	yd ²
km ²	square kilometers	0.4	square miles	mi ²
ha	hectares(10,000m ²)	2.5	acres	

MASS (weight)

g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000kg)	1.1	short tons	

VOLUME

ml	milliliters	8.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m ³	cubic meters	36	cubic feet	ft ³
m ³	cubic meters	1.3	cubic yards	yd ³

TEMPERATURE (exact)

°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F
°F	-40	0	32	40
°C	-20	0	32	40
		20	68	100
		40	104	200
		60	140	300
		80	176	400
		100	212	

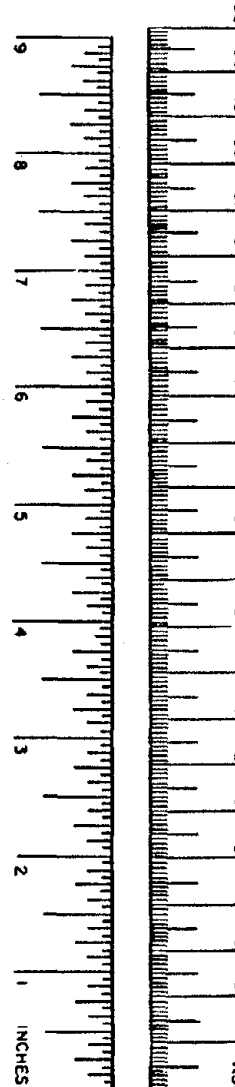


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- C. DETAILS OF FULL-SCALE CRASH TESTS
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Report No. FHWA/RD-82/074.2
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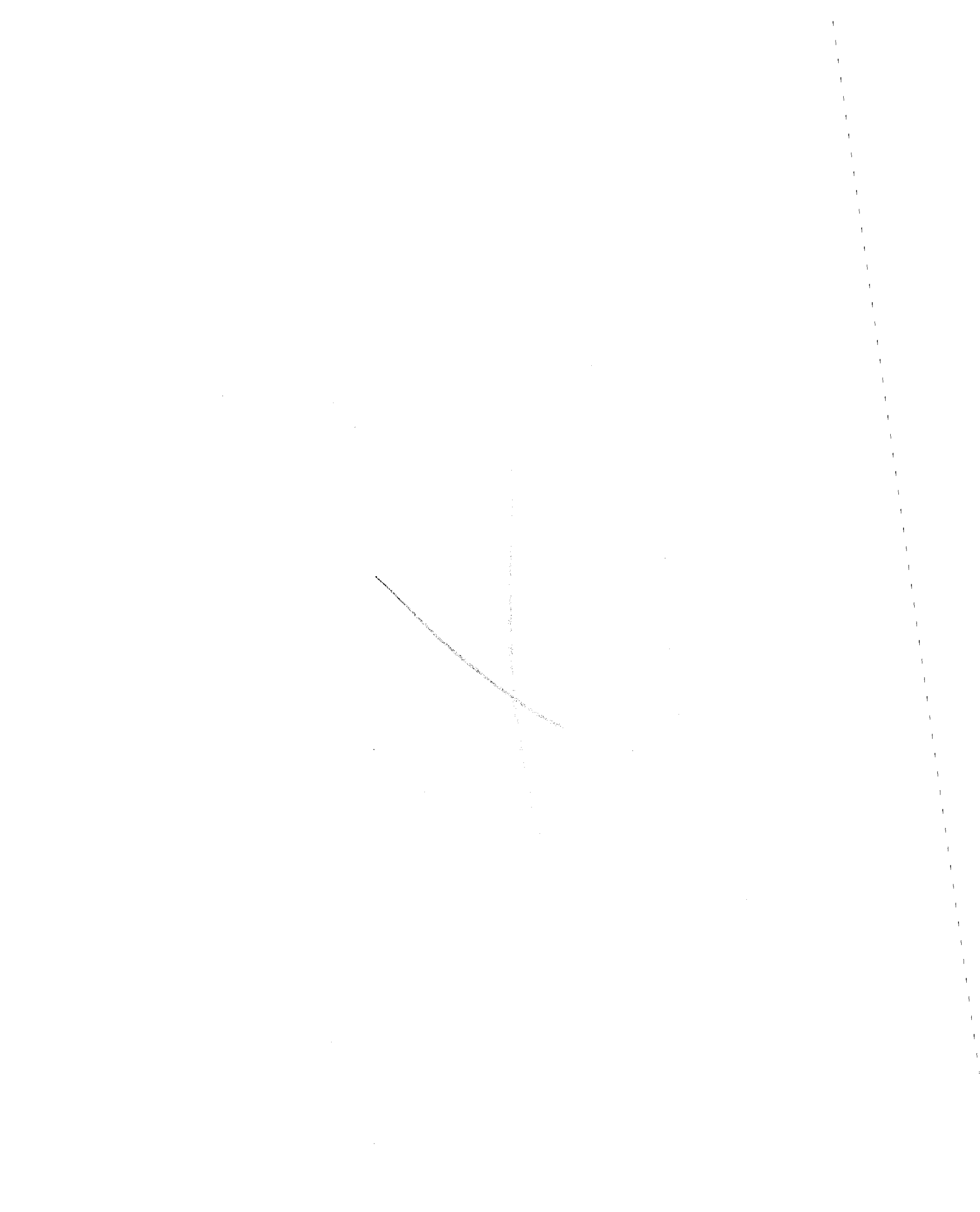
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I. INTRODUCTION

This report presents the results of a four-year study performed by the Texas Transportation Institute for the Federal Highway Administration. The study was initiated by the Federal Highway Administration in an effort to upgrade the performance of bridge railing systems.

The objectives of this study were:

1. to study and evaluate the performance of various bridge railing systems designed to meet the current (1976) American Association of State Highway and Transportation Officials, AASHTO, Specifications (17).
2. to develop recommended performance standards for bridge railing systems with consideration being given to vehicles ranging in size from an 863 kg (1,900 lb) subcompact automobile to a 14,528 kg (32,000 lb) intercity bus.
3. to develop recommended design guidelines and/or changes to the AASHTO bridge railing specifications that would allow one to design railing systems to meet the recommended performance standards.

The study began September, 1976, and was completed January, 1983. A total of 30 full-scale vehicle crash tests were conducted. Twenty-one of these were performed on five in-service railing designs and nine were performed on an instrumented wall.

The in-service designs were selected to critically examine specific clauses in the AASHTO Specifications (17). In the selection process, consideration was also given to the types of materials employed and to the style of designs being built under the specifications. Tests on the in-service designs showed that some designs failed to meet the desired performance level with automobiles while others performed adequately.

Tests were conducted on an instrumented wall to establish baseline values of performance measures and to establish design values for loads and load distributions.

This study did not address questions having to do with the manner in which railing loads are distributed into the deck or the required strength of the deck. Questions addressed were confined to the railing above the deck and all test railings were installed on massive rigid concrete foundations.

II. FULL-SCALE CRASH TESTS

Test Program

The test program, which included 30 full-scale crash tests, is summarized in Table 1. Testing extended over the period from January 1978 to October 1980 and was conducted generally in accordance with procedures recommended in Transportation Research Circular 191 (3).

Five bridge railing designs were selected for testing and evaluation. These designs were selected from those being used by the states at that time. Types of material, geometrics, and amount of railing being used were considered in the selection process. The railing designs selected were the following:

- Colorado Type 5 (steel)
- Texas T101 (steel)
- New Hampshire Two-Bar Aluminum
- North Carolina One-Bar Aluminum on Concrete Parapet
- Indiana Type 5A Aluminum and Modified Type 5A

In addition to these railing designs, a load measuring vertical concrete wall was constructed and used in a series of full-scale crash tests. Also, in one test, a modification of the Indiana Type 5A Aluminum design was used. In this modified design, the lower rail element was lowered so that it was centered 0.38 m (15 in.) (current specification lower limit) above the deck.

Geometrics of the cross sections of these railing systems are given in Figures 1 through 5 and in Figures 7 and 8. More complete detailed engineering drawings of these systems are given in Appendix C.

It was originally intended that only railings meeting the AASHTO Specifications (17) and being used at that time be considered for testing and evaluation. However, upon closer study, it was discovered that the geometry of many railing systems was not within the limits that specifications stated it "should" be. The "should" statement was not being interpreted as an absolute requirement and these railing systems, such as the Colorado Type 5, were being installed under the specifications. After this discovery, FHWA took action through a memorandum (22) to require that railing designs meet the "should" statement in the AASHTO Specifications (17).

Table 1. Summary of Full-Scale Crash Tests.

TEST NUMBER	TEST DATE	TEST CONDITIONS	RAILING DESIGN	COMMENTS
3451-01	01/04/78	2,770 lb/56.0 mph/15.1°	Colorado Type 5	Slight snagging
3451-02	01/05/78	4,700 lb/62.8 mph/15.0°	Colorado Type 5	Slight snagging
3451-03	01/09/78	4,640 lb/61.4 mph/24.5°	Colorado Type 5	Snagging
3451-04	02/23/78	19,760 lb/59.4 mph/14.3°	Colorado Type 5	Bus was contained, but rolled
3451-05	05/04/78	2,780 lb/57.3 mph/15.0°	Texas T101	Very smooth redirection
3451-06	05/09/78	4,660 lb/60.2 mph/15.0°	Texas T101	Very smooth redirection
3451-07	05/11/78	4,630 lb/59.8 mph/25.8°	Texas T101	Contained, rail deflected
3451-08	05/30/78	6,900 lb/53.4 mph/15.0°	Texas T101	Contained and redirected
3451-09	06/07/78	19,940 lb/55.3 mph/15.2°	Texas T101	Contained, unique behavior
3451-10	06/21/78	20,010 lb/52.0 mph/13.2°	Texas T101	Contained, unique behavior
3451-11	08/31/78	31,880 lb/58.4 mph/16.0°	Texas T101	Bus was contained, but rolled
3451-12	12/14/78	1,950 lb/60.9 mph/15.0°	New Hampshire	Severe snagging, car rolled
3451-13	12/05/78	2,780 lb/58.4 mph/15.0°	New Hampshire	Snagging
3451-14	01/15/79	2,780 lb/59.1 mph/20.5°	New Hampshire	Severe snagging
3451-15	12/12/78	4,670 lb/59.2 mph/15.0°	New Hampshire	Snagging
3451-23	02/16/79	19,920 lb/57.3 mph/14.8°	North Carolina	Bus was contained, but rolled
3451-24	06/26/79	1,950 lb/57.5 mph/12.5°	Indiana 5A	Very smooth redirection
3451-25	06/28/79	2,780 lb/53.6 mph/19.5°	Indiana 5A	Snagging
3451-26	07/03/79	4,670 lb/61.6 mph/25.8°	Indiana 5A	Vehicle penetrated railing
3451-27	07/31/79	2,150 lb/54.8 mph/20.0°	Indiana 5A	Snagging
3451-28	02/11/80	2,050 lb/55.4 mph/19.0°	Modif. Indiana 5A	Snagging
3451-29	04/23/80	1,970 lb/59.0 mph/15.5°	Instrumented Wall	Successful test
3451-30	04/15/80	2,800 lb/58.3 mph/14.8°	Instrumented Wall	Successful test
3451-31	04/18/80	2,830 lb/56.0 mph/20.0°	Instrumented Wall	Successful test
3451-32	04/29/80	4,680 lb/54.6 mph/16.5°	Instrumented Wall	Successful test
3451-33	05/01/80	4,700 lb/58.9 mph/23.8°	Instrumented Wall	Instrumentation failure
3451-34	05/06/80	20,030 lb/57.6 mph/16.5°	Instrumented Wall	Successful test
3451-35	10/24/80	32,020 lb/56.9 mph/15.8°	Instrumented Wall	Successful test
3451-36	05/20/80	4,740 lb/59.8 mph/24.0°	Instrumented Wall	Successful repeat of Test 33
3451-37	10/24/80	2,090 lb/58.5 mph/21.0°	Instrumented Wall	Successful test

1 lb = 0.454 kg

1 mph = 1.609 kph

Detailed engineering analyses of the allowable load capacities and ultimate strengths of these railing systems are presented in Appendices A and B. Summaries of these computed strengths are given in Tables 2 and 3. The allowable loads were calculated using the elastic analysis procedures specified in the AASHTO bridge specifications for 1976 and 1979 (17, 18). The controlling structural elements were determined and maximum allowable loads were calculated for each railing for two, three and four posts between splices in the rail element. Results of these analyses are summarized in Table 2 and details are presented on the following pages. Note that strength values given in Table 2 are for four posts between splices in rail elements. In some designs, shorter rail lengths are allowed.

The ultimate strength structural analysis were based on bending moments induced in the structure and the formation of plastic hinges at points of high bending moment. The failure mechanism and the number of posts involved in the mechanism are dependent upon the span-wise distribution of the load applied by the impacting vehicle. Several mechanisms for each railing system were investigated.

The validity of an ultimate strength, failure mechanism is dependent upon the ability of the structure to deform enough to actually develop the failure mechanism. In order for this to occur, sufficient plasticity must exist at points where hinges form to allow the formation of an adequate number of hinges and a mechanism. A determination of the ability of the railing structures to completely develop a failure mechanism was not made in the analyses presented. For the railing structures to resist the computed ultimate loads stated, sufficient rotation of the first plastic hinges to form would be necessary. For the two-span and three-span mechanisms, the first plastic hinge to form is in the post. Since this structural element is relatively short compared to the span length of the rail element, a comparatively large amount of plastic rotation in the hinge at the base of the post would be required for complete development of plastic hinges in the rail elements. It is doubtful that this can occur, especially in the aluminum railing systems, because of the nature of the connection between the post and deck (rivets, baseplate and sail). It is quite probable that a progressive failure (first the posts, then the rail

Table 2. Computed Allowable Loads for Railings (kips)

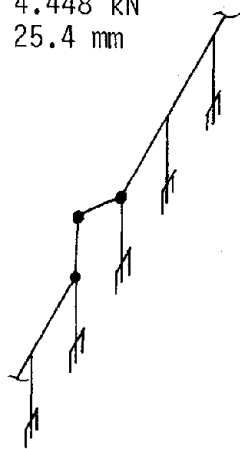
Failure Mode \ Railing Design	Colorado Type 5	Texas T101	New Hampshire	North Carolina	Indiana 5A
Bending of top rail element	20.4	10.5	8.2	5.2	4.2
Bending of second rail element	----	----	8.2	----	4.2
Concrete parapet	----	----	---	22.2	---
Sum of all rail elements	20.4	10.5	16.4	27.4	8.4
Bending of post	11.3	10.2	9.5	5.9	9.0
Post-to-baseplate connection	13.6	10.2	7.9	4.6	7.3
Baseplate Thickness	3.5	8.2	---	---	---
Baseplate Bearing	8.4	4.6	6.9	4.1	6.4
Baseplate/Sail	---	---	1.4	4.8	1.3
Anchor bolts	8.9	8.7	6.9	4.2	6.5
Meets AASHTO strengths rqmts	No	No	No	No	No
Meets AASHTO geometric rqmts	No	Yes	Borderline	Yes	Yes

1 kip = 4.448 kN

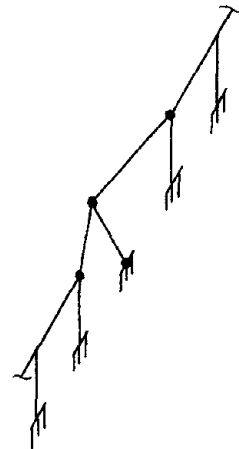
Table 3. Computed Ultimate Strength for Each Type of Bridge Railing.

RAIL TYPE	ULTIMATE STRENGTH AND LOCATION OF RESULTANT		
	1 SPAN MECHANISM (kips and in.)	2 SPAN MECHANISM (kips and in.)	3 SPAN MECHANISM (kips and in.)
Colorado Type 5	77.1/27.0	64.9/27.0	84.1/27.0
Texas T101	32.4/21.0	40.2/21.0	61.0/21.0
New Hampshire Two-Bar	39.5/30.5	33.7/30.5	43.9/30.5
North Carolina One-Bar	120.6/20.6	93.5/21.2	----
Indiana Type 5A	43.3/23.7	33.5/23.7	42.0/23.7

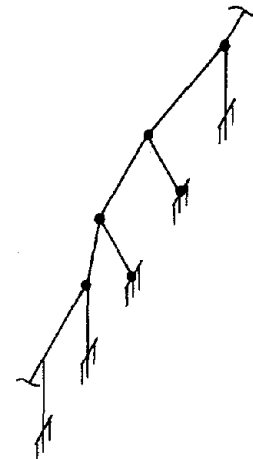
1 kip = 4.448 kN
1 in. = 25.4 mm



One-Span Mechanism



Two-Span Mechanism



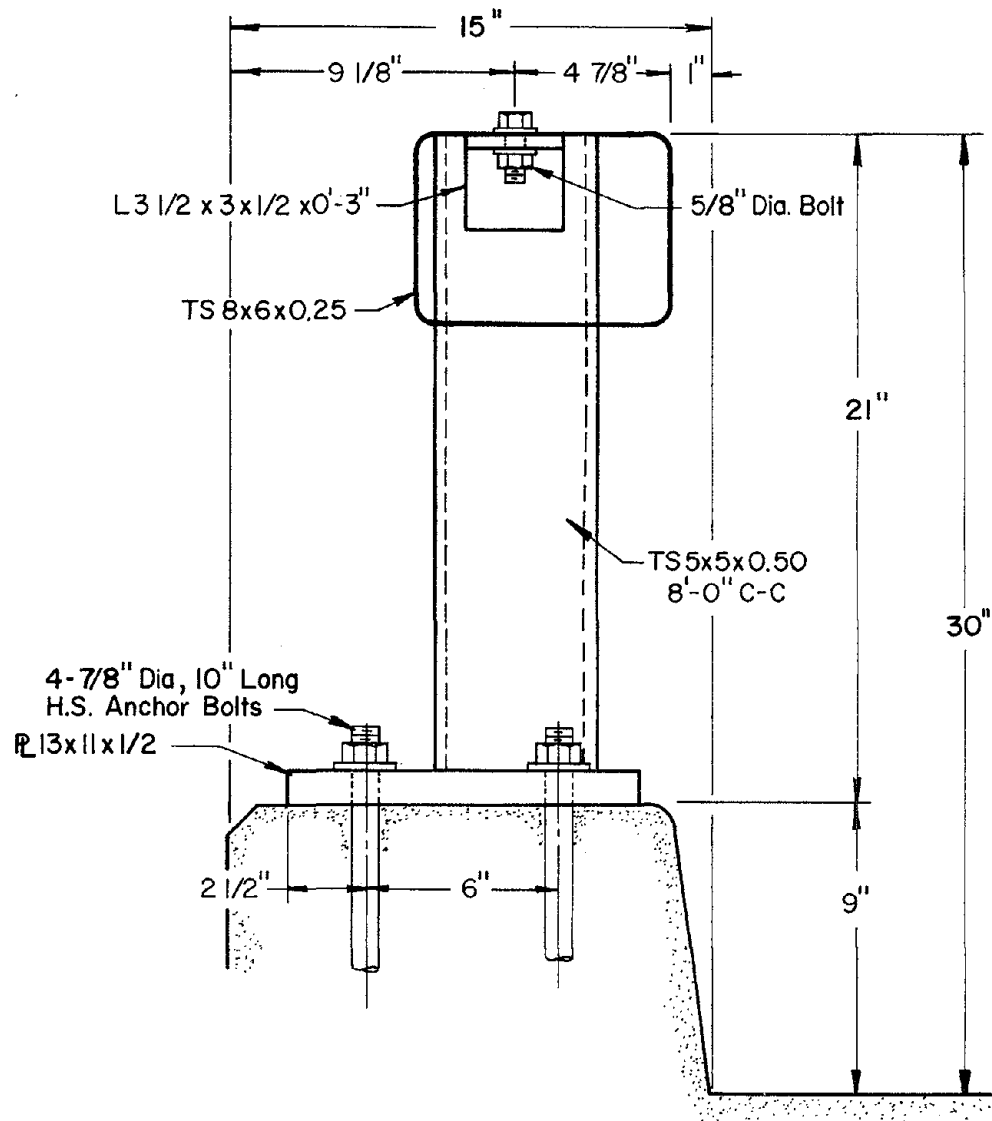
Three-Span Mechanism

element) at a reduced load would occur in these railing systems. In order to provide a reasonable degree of assurance that the computed ultimate loads could be achieved, one would need to perform a detailed displacement analysis and possibly modify the connections in these railing systems.

The Colorado Type 5 railing was selected for the most part, on the basis of the geometry it presented (Figure 1). The cross section consists of a 0.23 m (9 in.) high curb, a 0.38 m (15 in.) high opening immediately above the curb and a 0.15 m (6 in.) high face of a tubular rail element. This cross section does not provide ". . . a rail centered between 15 and 20 in. above the referenced surface." The 0.38 m (15 in.) high vertical opening between the curb and the lower side of the metal rail element presented a potential for snagging of a vehicle wheel. However, the severity of snagging that might occur and the contribution of the curb to redirection of a vehicle were in need of investigation. The railing system was being allowed under the AASHTO Specifications (17) at that time. A strength analysis of this railing indicated that it basically satisfied requirements of the specifications with the baseplate and anchor bolts being marginal.

The Texas T101 railing system (Figure 2) was considered to meet both the strength and geometric requirements of the AASHTO Specifications (17). Total height of this railing was 0.65 m (27 in.) which is the minimum permitted by AASHTO specifications. The rail element was a corrugated sheet steel beam (AASHTO M-180) strengthened with two tubular steel members. Because of the widespread use of corrugated sheet steel beams in railing systems, it was deemed necessary to include one such railing design in the testing program. Performance of this railing in full-scale automobile tests was expected to be adequate; however, the upper limit of performance that it might achieve with larger vehicles was not known. It was subjected to a series of seven tests with vehicles ranging from a Vega [1,022 kg (2,250 lb)] to an intercity bus [14,528 kg (32,000 lb)]. A Honda [817 kg (1,800 lb)] was not used in this series.

The New Hampshire Two-Bar Aluminum railing system is mounted on an 0.22 m (8 1/2 in.) high curb that projects 0.23 m (9 in.) from the traffic side of the metal railing (Figure 3). The use of such curbs has been common in some states in the past and is still allowed by AASHTO Specifi-

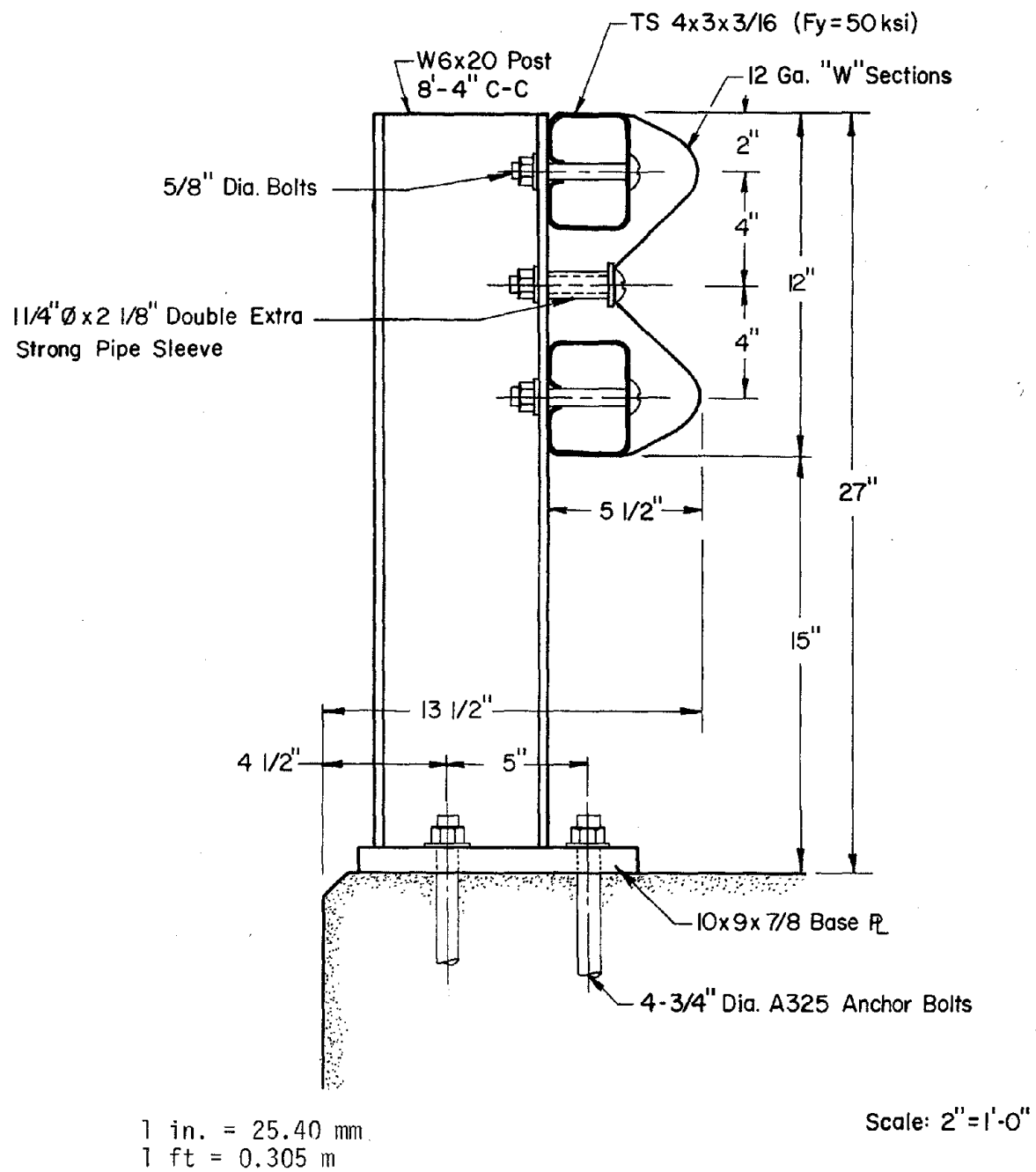


Scale: 2" = 1'-0"

1 in. = 25.40 mm
1 ft = 0.305 m

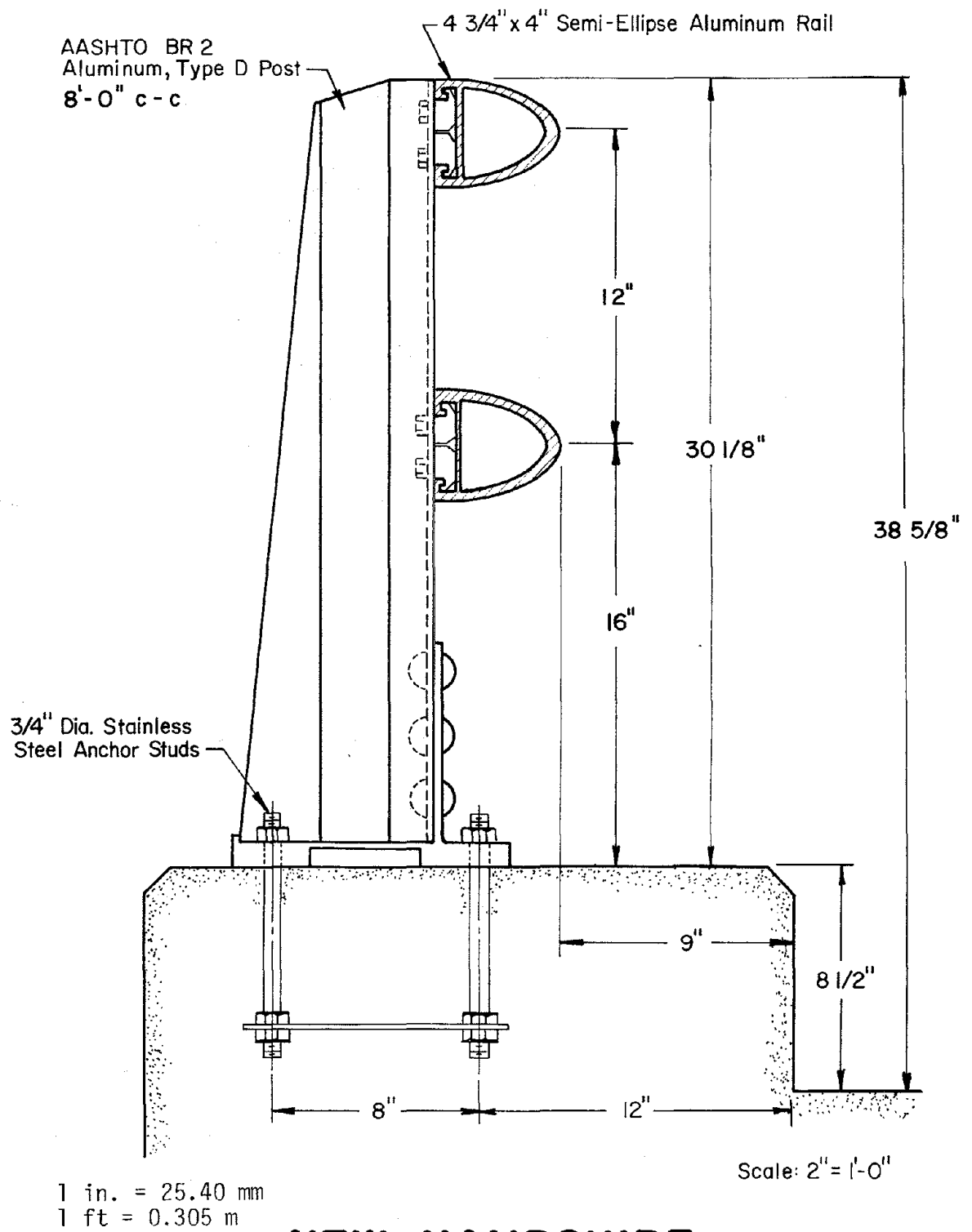
COLORADO TYPE 5

Figure 1. Cross Section of Colorado Type 5 Railing.



T 101

Figure 2. Cross Section of Texas T101 Railing.



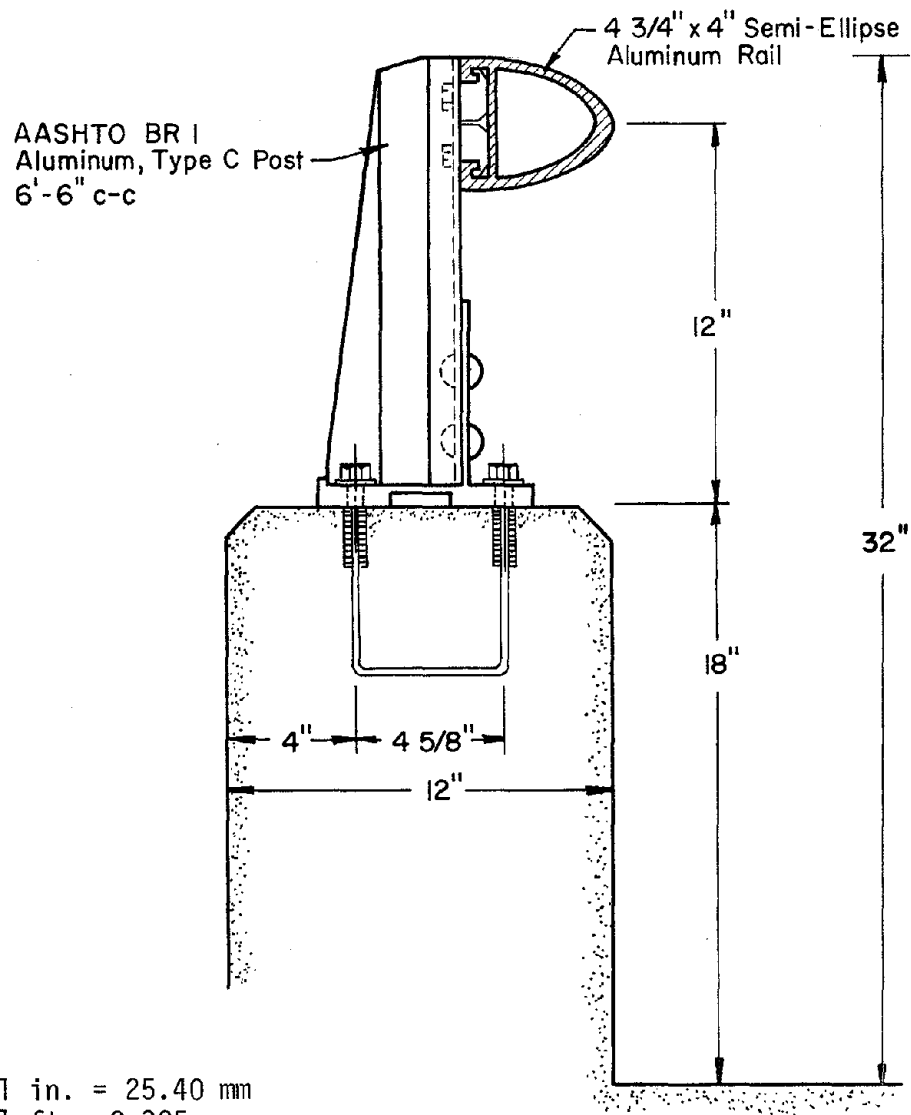
NEW HAMPSHIRE

Figure 3. Cross Section of New Hampshire Two-Bar Aluminum Railing.

cations. The reference surface for this railing system is the upper surface of the deck (curb projects 0.23 m (9 in.) or less) and the lower rail element is centered at 0.62 m (24 1/2 in.) above the deck. This does not meet the Specification (17) statement of "... a rail centered between 15 and 20 in. above the surface." If the curb projected more than 0.23 m (9 in.) from the traffic face of the railing the reference surface would be the top of the curb and the railing geometry would meet specifications. Also, this design makes use of structural elements that are widely used in railing designs constructed of aluminum. It was subjected to a series of four automobile tests.

The North Carolina One-Bar Aluminum railing on concrete parapet was selected for somewhat unique reasons. Many states use a design consisting of an 0.46 m (18 in.) high concrete parapet with some type of metal railing on top (Figure 4). The concrete parapet was originally the key feature involved in selection of this railing system. However, one of the objectives of this study was to define the performance of existing railing systems in impacts with heavier vehicles. This railing design was finally selected to represent concrete parapet designs but because of limitations on the testing program, the only test conducted on it was with a 9,080 kg (20,000 lb) school bus at 97 kph (60 mph) and 15 deg.

The Indiana Type 5A Aluminum railing uses metal elements similar to the New Hampshire design except no curb is involved in the Indiana system (Figure 5). Selection of this design was made after question about the influence of the curb had been generated through testing of the New Hampshire system. The Indiana design meets geometric requirements of the AASHTO Specifications (17) and it was thought at the time of its selection to almost meet strength requirements. However, in the full-scale testing program, a previously unanticipated failure mode in the baseplate of the post was discovered. This failure mode was attributed to the reduced thickness in the middle portion of the baseplate. Instead of functioning as a stiff plate and loading the vertical sail in mostly uniform tension, the toe of the baseplate bent downward, under load, and caused very high combined tensile and bending stresses to occur in the vertical sail near its juncture with the baseplate (Figure 6). This resulted in a drastically reduced post strength. A detailed strength analysis of this failure mode



1 in. = 25.40 mm
1 ft = 0.305 m

Scale: 2" = 1'-0"

NORTH CAROLINA

Figure 4. Cross Section of North Carolina One-Bar Aluminum on Concrete Parapet.

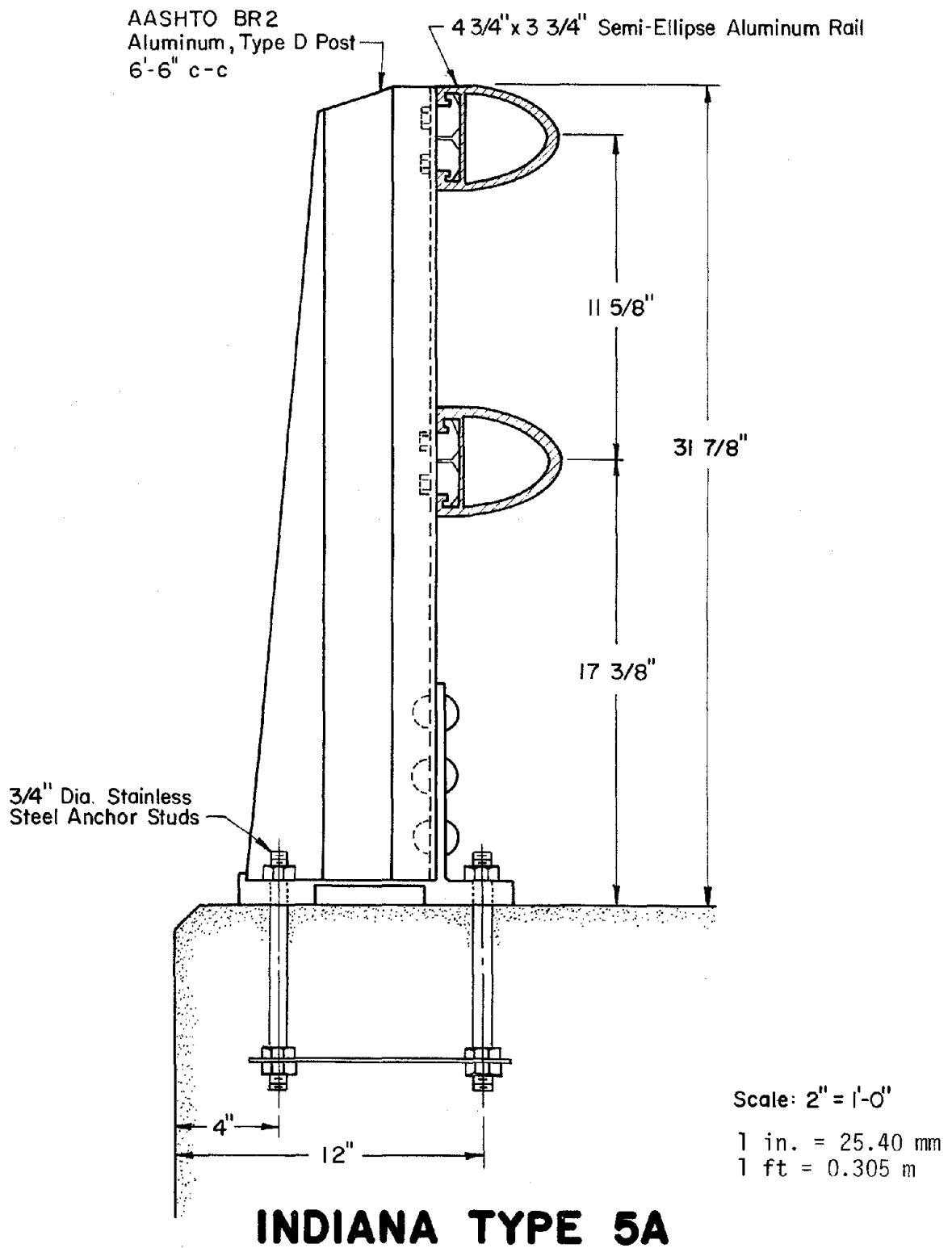


Figure 5. Cross Section of Indiana Type 5A Aluminum Railing.

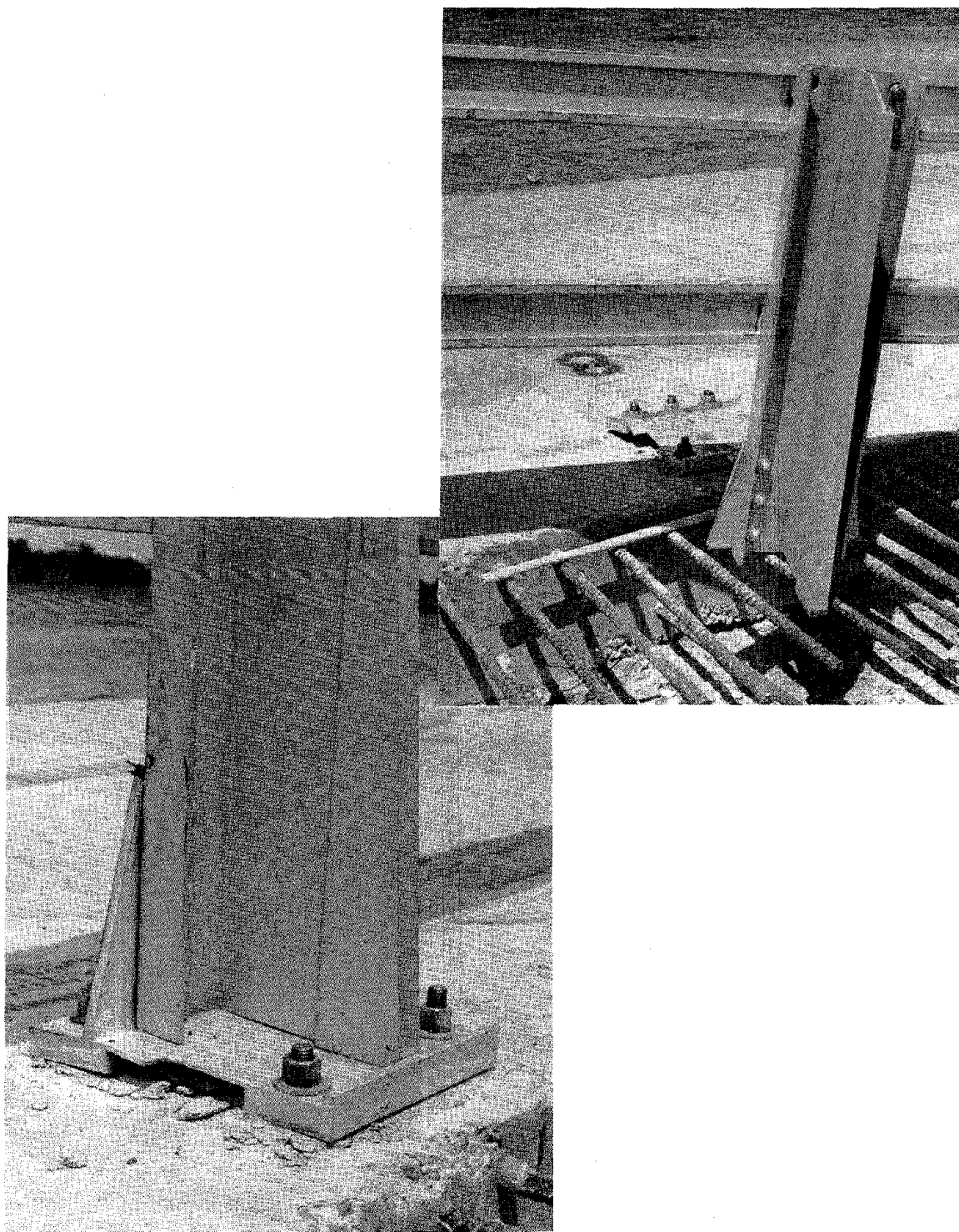


Figure 6. Failure Mode in Indiana Baseplate.

is presented in Appendix A.

The Modified Type 5A railing (Figure 7) was selected for testing in order to evaluate the adequacy of geometric requirements in the specifications (17). This railing design was identical to the Indiana Type 5A design except the lower rail element was lowered such that it was centered 0.38 m (15 in.) above the deck. Snagging of the front wheel had occurred in tests on the Indiana Type 5A with the Honda and Vega. With both vehicles, the front wheel underrode the lower rail element and snagged on posts. The modified design was intended to preclude snagging behavior to the extent possible by modifying rail placement within the limits of the specifications.

A series of nine full-scale crash tests was performed on the lateral load measuring instrumented wall whose cross-section is shown in Figure 8. Test vehicle size ranged from the 817 kg (1,800 lb) Honda Civic to the 14,528 kg (32,000 lb) intercity bus. Detailed descriptions of the geometrics of this installation, its functional features and calibrations for measurement of loads are presented in Appendix C.

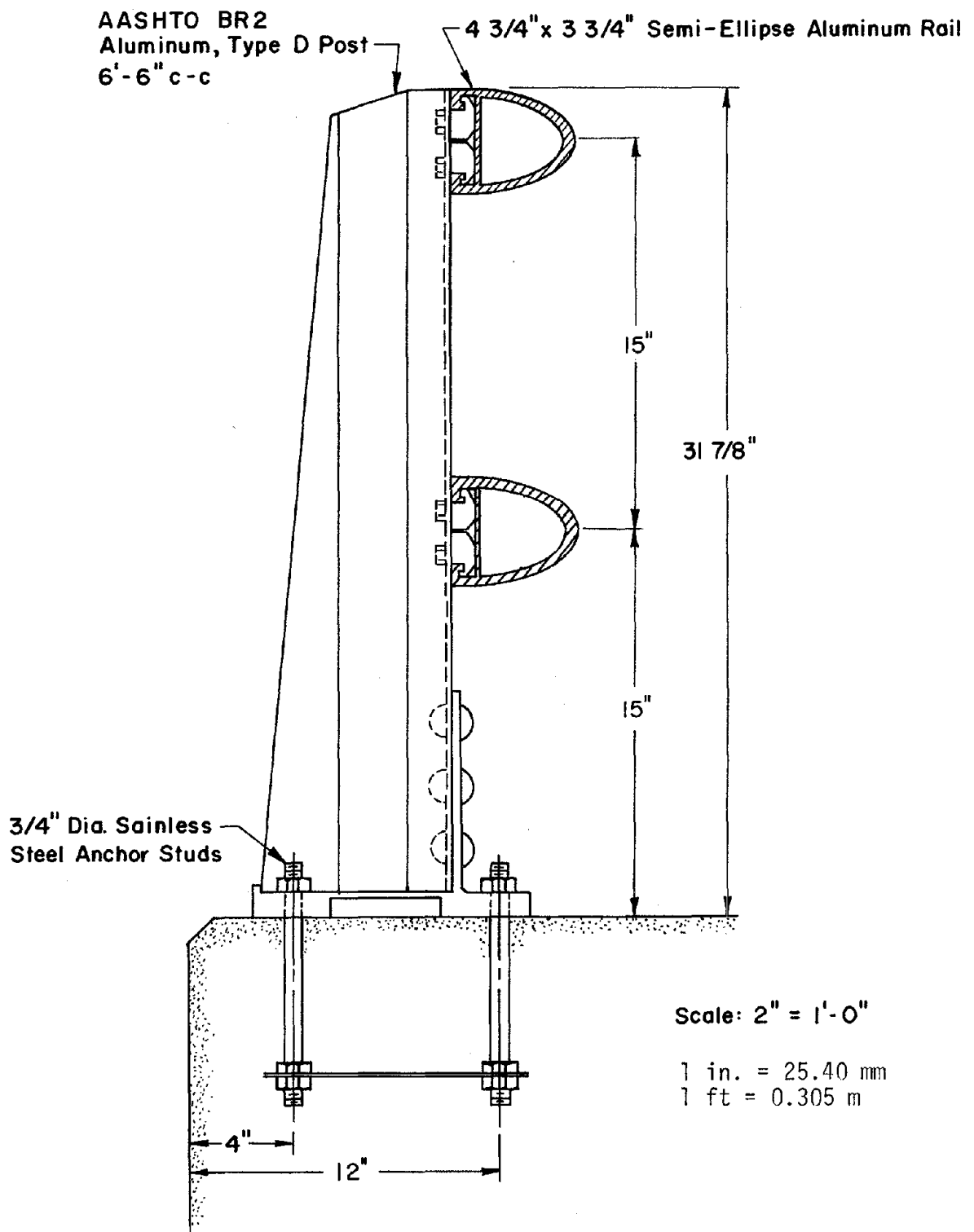
The purpose of the series of tests on the instrumented wall was to gather experimental information on magnitudes, distributions and locations of resultants of lateral forces imposed during collisions, and to make measurements of baseline vehicle responses for use in establishing performance standards for railing systems.

Transportation Research Circular 191 (3) addresses performance of railing systems for two sizes of vehicles--a 1,022 kg (2,250 lb) and a 2,043 kg (4,500 lb) automobile. A part of the objective of this study was to address design and performance of bridge railing systems for a larger range of vehicle sizes. Test vehicles used in this program were:

- Honda Civic - 817 kg (1,800 lb)
- Chevrolet Vega - 1,022 kg (2,250 lb)
- Plymouth Fury - 2,043 kg (4,500 lb)
- GMC 16-Passenger School Bus - 3,178 kg (7,000 lb)
- Ford/Wayne 66-Passenger School Bus - 9,080 kg (20,000 lb)
- GM PD4106 Intercity Bus - 14,528 kg (32,000 lb)

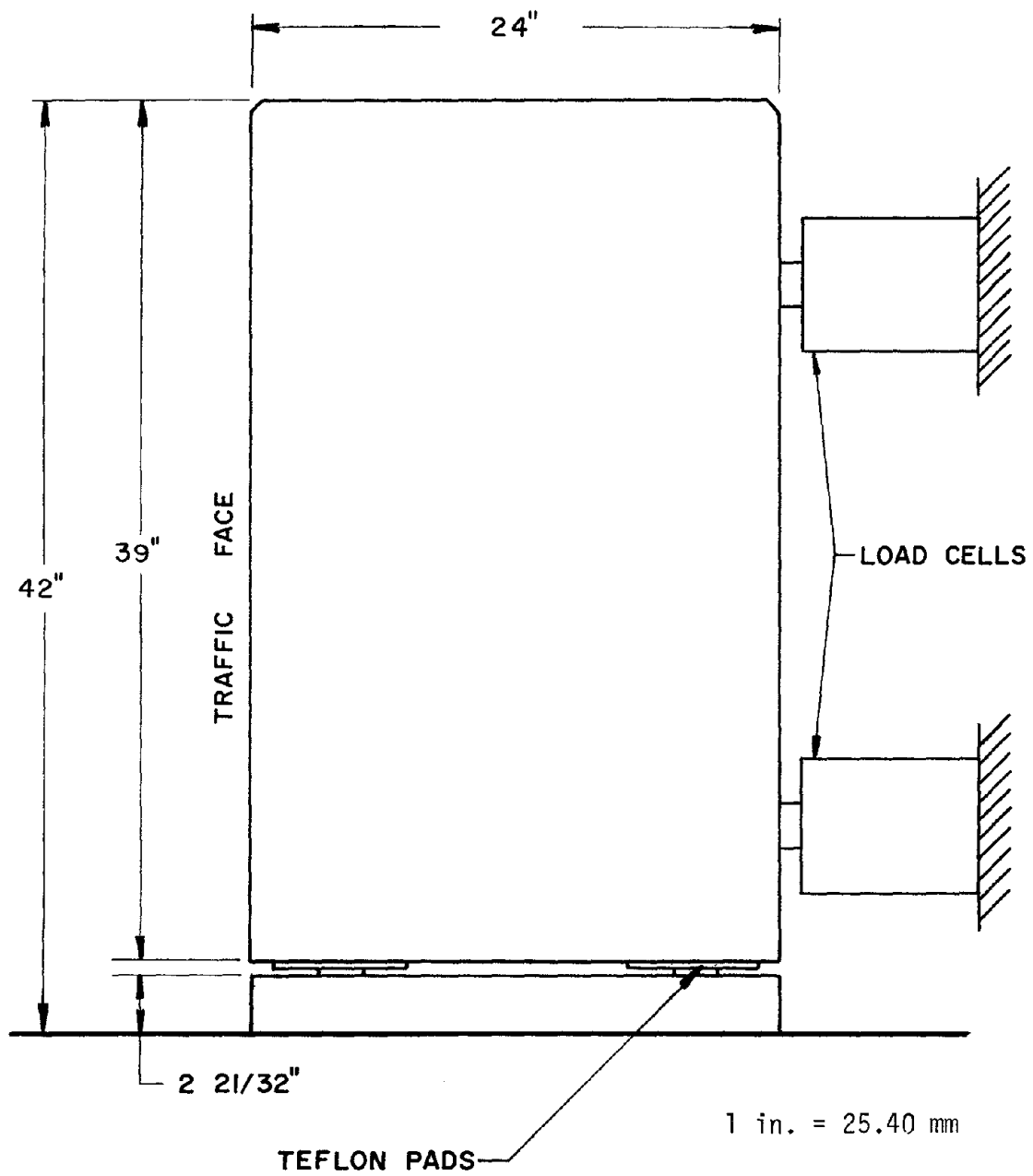
These vehicles are described in further detail in Chapter III.

Shadow drawings showing comparisons of vehicle and railing geometrics



MODIFIED INDIANA TYPE 5A

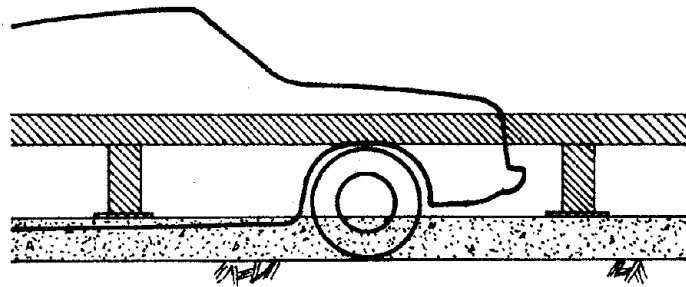
Figure 7. Cross Section of Modified Indiana Type 5A Railing.



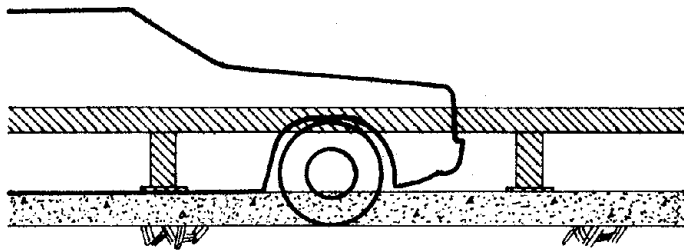
INSTRUMENTED WALL

Figure 8. Cross Section of Instrumented Wall.

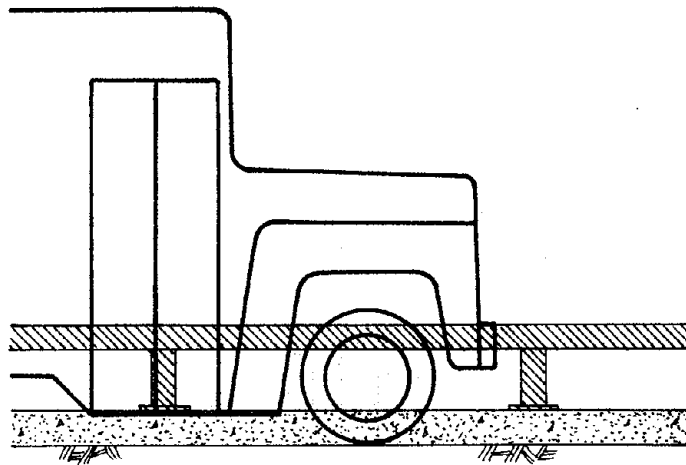
for the combinations tested are presented in Figures 9 through 13.



1974 Vega

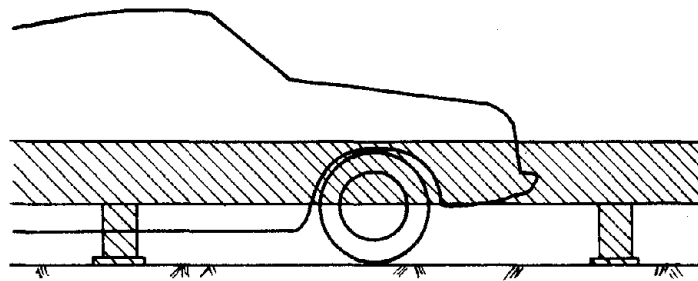


1974 Plymouth Fury

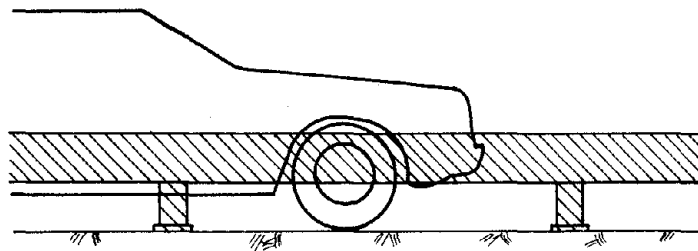


1969 Ford/Wayne 66-Passenger Bus

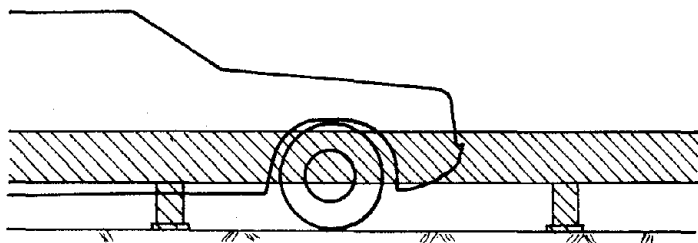
Figure 9. Geometrics of Vehicles and Colorado Type 5 Rail.



1974 Vega

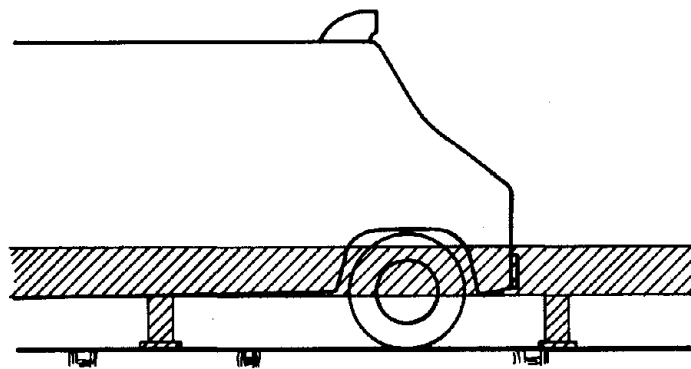


1974 Plymouth Fury

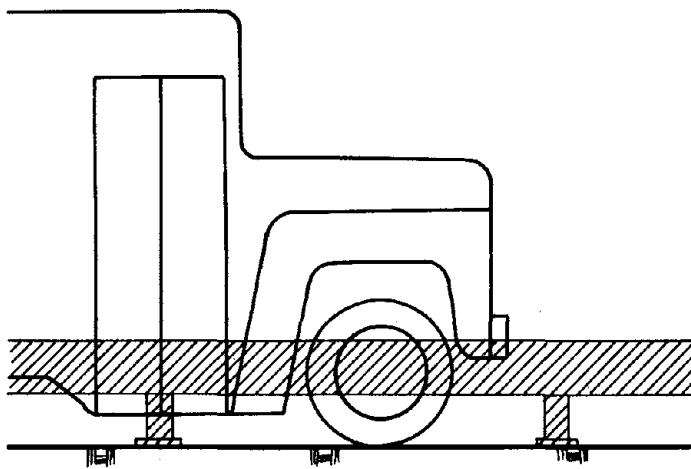


1976 Plymouth Fury

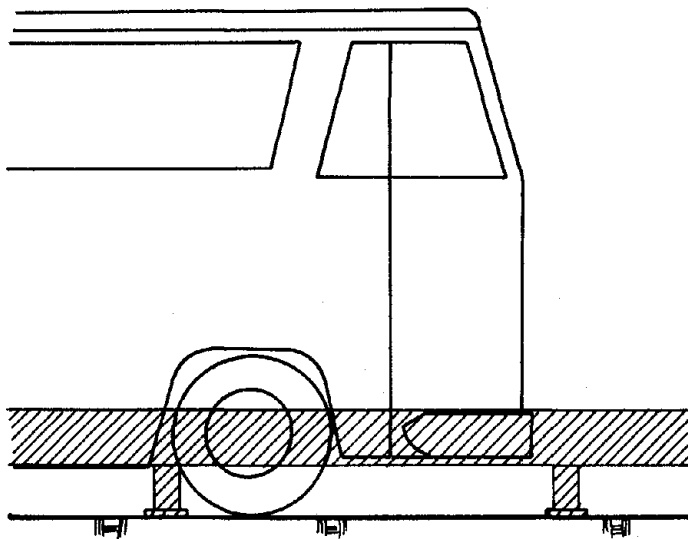
Figure 10. Geometrics of Vehicles and Texas T101 Rail.



1972 16-Passenger GMC Rally Wagon

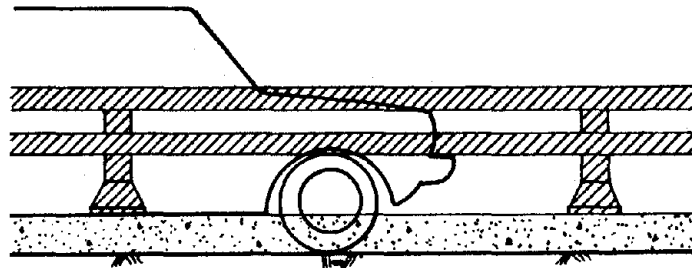


1969 Ford/ Wayne 66-Passenger Bus

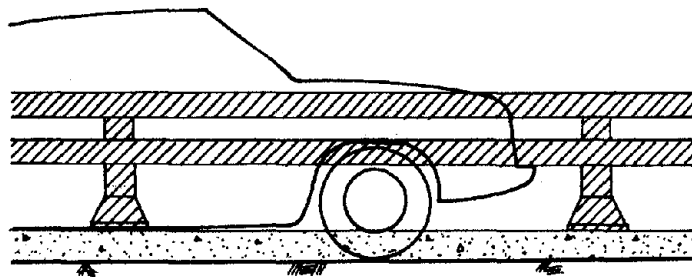


1962 GM #PD4106 45-Passenger Intercity Bus

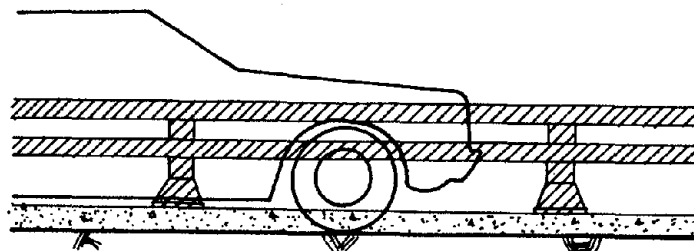
Figure 10. Geometrics of Vehicles and Texas T101 Rail. (Continued)



1974 Honda

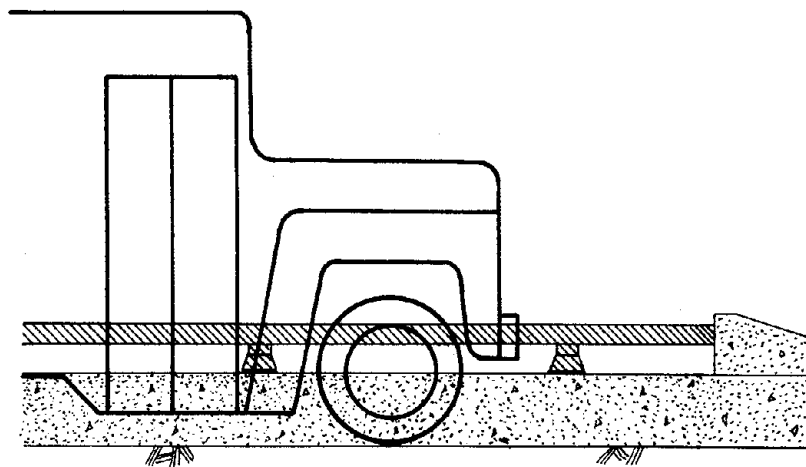


1974 Vega



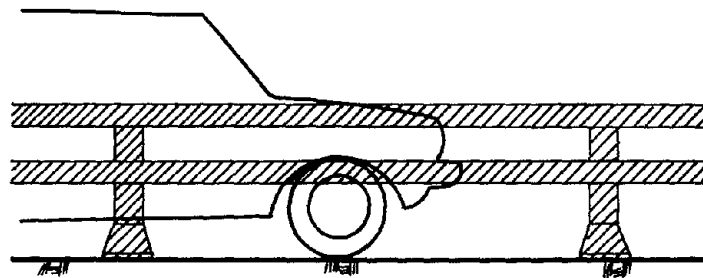
1974 Plymouth Fury

Figure 11. Geometrics of Vehicles and New Hampshire Two-Bar Aluminum Rail.

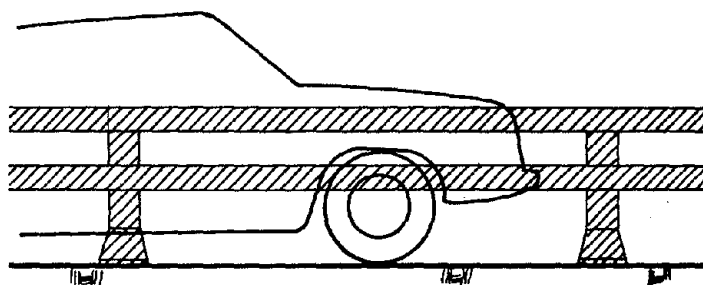


1969 Ford/Wayne 66-Passenger Bus.

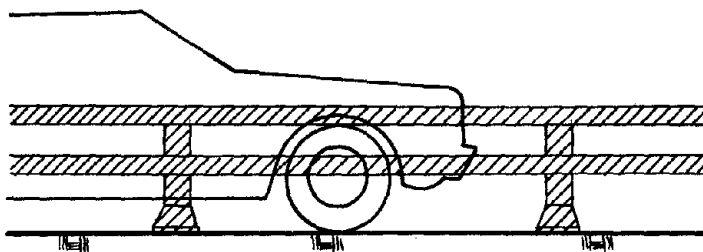
Figure 12. Geometrics of Vehicle and North Carolina One-Bar Aluminum Rail.



1974 Honda



1974 Vega



1974 Plymouth Fury

Figure 13. Geometrics of Vehicles and Original Configuration of Indiana Type 5A Railing.

General Test Results

Tables 4 and 5 summarize some of the key results of each test in the program and Figures 14 through 78 give further details for each test. Detailed test results are given in Appendix C. Test results shown in Table 4 include those measures necessary for evaluating performance of a railing in accordance with Transportation Research Circular 191 (3). Evaluation criteria for structural adequacy do not provide for quantitative measures but are evaluated qualitatively. The last column in this table shows a determination of acceptability for each test addressed by TRC 191. For other tests "N. App." is shown to indicate that TRC 191 requirements are not applicable. Dummy response information is shown in the table but was not considered in determining acceptability of performance. It should be noted that only three tests (numbers 7, 33 and 36) are considered to meet the performance requirements. All of these were strength tests with 2,043 kg (4,500 lb) automobiles and two of them were performed on the instrumented wall. A frequent cause for failure to meet requirements was excessive lateral acceleration imposed on the vehicle.

Table 5 gives a summary of acceptability of performance based on criteria in NCHRP Report 230 (9). For determining acceptability by evaluation criterion "F", only the occupant/compartment impact velocity and not the ridedown accelerations were used because these tests were originally performed under Transportation Research Circular 191 and ridedown accelerations were not computed. It is noted that determinations of acceptability by evaluation criteria "A", "E" and "H" are judgement decisions because quantitative measures for determining acceptability are not provided in NCHRP Report 230.

Table 4. Summary of Test Results and Acceptability of Performance
by Transportation Research Circular 191. (3)

TRC 191 RQM'T	STRUCTURAL ADEQUACY			IMPACT SEVERITY MAX 50 msc AVG VEHICLE ACC			DUMMY RESPONSE (Optional)				ACCEPTABILITY OF PERFORMANCE BY TRC 191
	SHALL NOT POCKET OR SNAG	SHALL CONTAIN AND REDIRECT	INTEGRITY OF OCCUPANT COMPARTMENT MAINTAINED	LAT (5 g's)	LONG (10 g's)	TOTAL (12 g's)	DRIVER HIC (1000)	DRIVER CHEST (60)	PASS. HIC (1000)	PASS. CHEST (60)	
TEST NO.											
3451-1	Fail	Pass	Pass	7.2	-5.8	7.2	64	19	308	12	Fail
3451-2	Fail	Pass	Pass	5.8	-4.8	6.3	224	16	451	32	Fail
3451-3	Fail	Pass	Fail	7.1	-14.8	16.6	1125	40	435	36	Fail
3451-4	Fail	Pass	Fail	4.7	-1.6	4.8	76	13	371	9	N. App
3451-5	Pass	Pass	Pass	7.3	-2.1	7.6	215	17	368	15	Fail
3451-6	Pass	Pass	Pass	5.9	-2.8	7.2	351	16	351	33	Fail
3451-7	Pass	Pass	Pass	6.9	-5.2	7.9	409	21	139	27	Pass
3451-8	Pass	Pass	Fail	4.4	-2.8	5.4	87	9	65	6	N. App
3451-9	Fail	Pass	Pass	4.2	-1.6	4.2	54	23	213	30	N. App
3451-10	Pass	Pass	Pass	3.5	-1.6	3.7	46	27	178	22	N. App
3451-11	Fail	Pass	Fail	2.0	-1.2	2.1	37	10	119	13	N. App
3451-12	Fail	Fail	Fail	9.5	-12.8	15.1	867	23	237	17	Fail
3451-13	Fail	Pass	Fail	9.0	-7.1	10.9	298	21	96	15	Fail
3451-14	Fail	Pass	Fail	8.9	-17.1	17.8	1088	39	1135	41	N. App
3451-15	Fail	Pass	Pass	6.0	-5.1	7.2	97	12	108	23	Fail
3451-23	Fail	Fail	Fail	4.9	-1.9	5.1	50	8	143	37	N. App
3451-24	Pass	Pass	Pass	8.9	-2.3	9.2	256	25	82	18	Fail
3451-25	Fail	Pass	Fail	6.6	-4.9	9.0	---	36	75	20	N. App
3451-26	Fail	Fail	Pass	6.5	-14.8	15.1	383	38	214	22	Fail
3451-27	Fail	Pass	Fail	10.3	-9.2	13.6	413	54	93	37	N. App
3451-28	Fail	Pass	Fail	10.2	-13.6	14.8	692	43	181	40	N. App
3451-29	Pass	Pass	Pass	10.3	-4.0	11.0	159	46	143	44	Fail
3451-30	Pass	Pass	Pass	7.7	-3.0	8.2	100	25	119	19	Fail
3451-31	Pass	Pass	Pass	8.2	-3.7	8.7	588	33	80	17	N. App
3451-32	Pass	Pass	Pass	9.3	-4.0	10.1	510	21	101	24	Fail
3451-33	Pass	Pass	Pass	---	---	---	---	--	---	--	Pass
3451-34	Pass	Pass	Pass	6.3	-1.9	6.4	130	19	39	20	N. App
3451-35	Pass	Pass	Pass	8.6	-1.4	8.7	79	15	70	28	N. App
3451-36	Pass	Pass	Pass	15.5	-9.1	17.7	1228	29	130	23	Pass
3451-37	Pass	Pass	Pass	13.1	-6.5	14.6	230	20	216	50	N. App

Table 5. Summary of Acceptability of Performance by NCHRP Report 230. (9)

TEST NUMBER	TEST DATE	TEST CONDITIONS	NCHRP REPORT 230 PERFORMANCE CRITERIA						ACCEPTABILITY
			A	D	E	F	H	I	
3451-01	01/04/78	2,770 lb/56.0 mph/15.1°	Pass	Pass	Pass	Pass	Pass	Pass	Pass
3451-02	01/05/78	4,700 lb/62.8 mph/15.0°	Pass	Pass	Pass	--	Pass	Fail	Fail
3451-03	01/09/78	4,640 lb/61.4 mph/24.5°	Fail	Pass	Pass	--	Pass	Fail	Fail
3451-04	02/23/78	19,760 lb/59.4 mph/14.3°	Pass	Pass	Fail	--	--	--	Fail
3451-05	05/04/78	2,780 lb/57.3 mph/15.0°	Pass	Pass	Pass	Pass	Pass	Pass	Pass
3451-06	05/09/78	4,660 lb/60.2 mph/15.0°	Pass	Pass	Pass	--	Pass	Pass	Pass
3451-07	05/11/78	4,630 lb/59.8 mph/25.8°	Pass	Pass	Pass	--	Pass	Fail	Fail
3451-08	05/30/78	6,900 lb/53.4 mph/15.0°	--	--	--	--	--	--	--
3451-09	06/07/78	19,940 lb/55.3 mph/15.2°	Pass	Pass	Pass	--	--	--	Pass
3451-10	06/21/78	20,010 lb/52.0 mph/13.2°	Pass	Pass	Pass	--	--	--	Pass
3451-11	08/31/78	31,880 lb/58.4 mph/16.0°	Pass	Pass	Fail	--	--	--	Fail
3451-12	12/14/78	1,950 lb/60.9 mph/15.0°	Fail	Pass	Fail	Pass	Pass	?	Fail
3451-13	12/05/78	2,780 lb/58.4 mph/15.0°	Pass	Pass	Pass	Pass	Pass	Fail	Fail
3451-14	01/15/79	2,780 lb/59.1 mph/20.5°	--	--	--	--	--	--	--
3451-15	12/12/78	4,670 lb/59.2 mph/15.0°	Pass	Pass	Pass	--	Pass	Pass	Pass
3451-23	02/16/79	19,920 lb/57.3 mph/14.8°	Pass	Pass	Fail	--	--	--	Fail
3451-24	06/26/79	1,950 lb/57.5 mph/12.5°	Pass	Pass	Pass	Pass	Pass	Pass	Pass
3451-25	06/28/79	2,780 lb/53.6 mph/19.5°	--	--	--	--	--	--	--
3451-26	07/03/79	4,670 lb/61.6 mph/25.8°	Fail	Pass	Pass	--	Pass	Fail	Fail
3451-27	07/31/79	2,150 lb/54.8 mph/20.0°	Fail	Pass	Fail	--	Pass	Pass	Fail
3451-28	02/11/80	2,050 lb/55.4 mph/19.0°	Fail	Pass	Fail	--	Pass	Fail	Fail
3451-29	04/23/80	1,970 lb/59.0 mph/15.5°	Pass	Pass	Pass	Pass	Pass	Pass	Pass
3451-30	04/15/80	2,800 lb/58.3 mph/14.8°	Pass	Pass	Pass	Pass	Pass	Pass	Pass
3451-31	04/18/80	2,830 lb/56.0 mph/20.0°	--	--	--	--	--	--	--
3451-32	04/29/80	4,680 lb/54.6 mph/16.5°	Pass	Pass	Pass	--	Pass	Pass	Pass
3451-33	05/01/80	4,700 lb/58.9 mph/23.8°	Pass	Pass	Pass	--	Pass	Pass	Pass
3451-34	05/06/80	20,030 lb/57.6 mph/16.5°	Pass	Pass	Pass	--	--	--	Pass
3451-35	10/24/80	32,020 lb/56.9 mph/15.8°	Pass	Pass	Pass	--	--	--	Pass
3451-36	05/20/80	4,740 lb/59.8 mph/24.0°	Pass	Pass	Pass	--	Pass	Fail	Fail
3451-37	10/24/80	2,090 lb/58.5 mph/21.0°	Pass	Pass	Pass	--	Pass	Fail	Fail

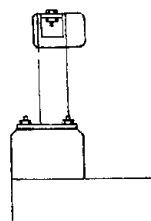
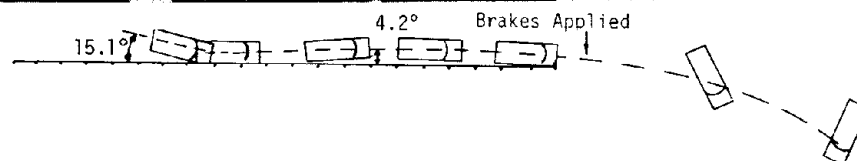
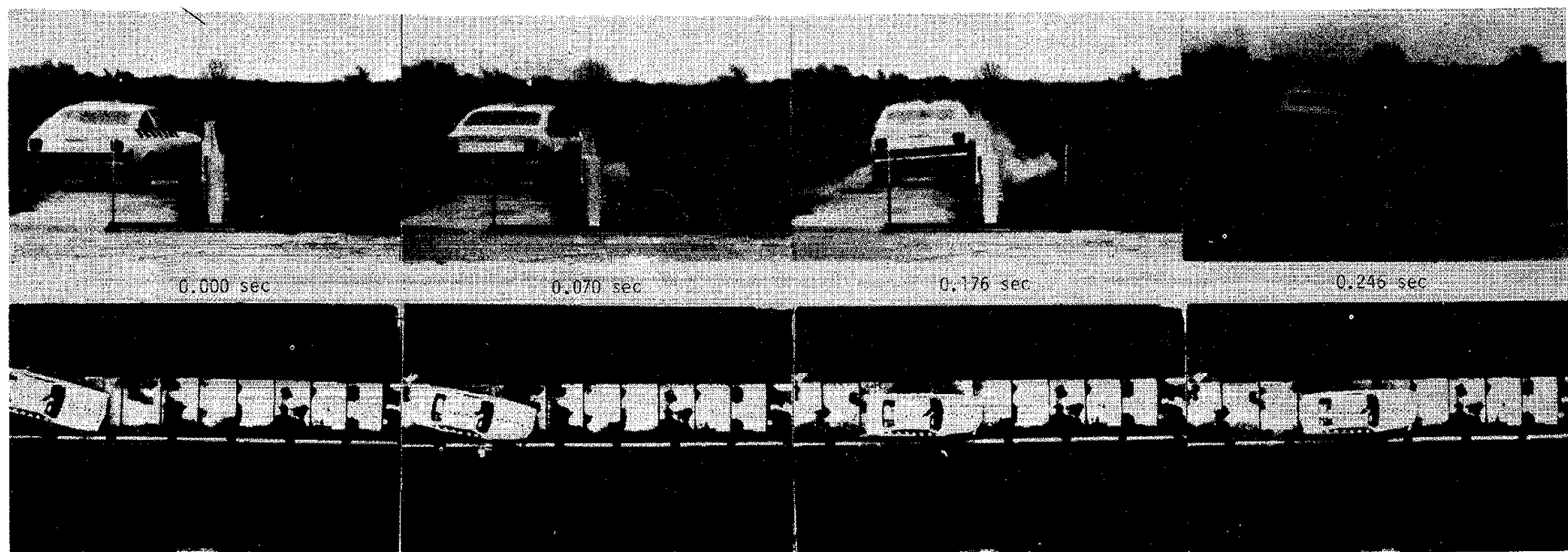
1 lb = 0.454 kg

1 mph = 1.609 kph

Results for Colorado Type 5 Railing: A series of four tests (Number 1 through 4) was conducted on the Colorado Type 5 railing. It was found that the design has undesirable geometrics in that the 0.38 m (15 in.) high open space between the curb and metal railing extends from 0.23 to 0.61 m (9 in. to 24 in.) above the roadway surface. This open space allowed excessive penetration of the automobile bumper and front wheel with subsequent snagging on the post, especially in the 25 deg test (Test 3).

The strength of the railing was adequate to prevent penetration of the school bus in a 15 deg impact. However, the 0.76 m (30 in.) total height of the railing was not adequate to provide sufficient roll stability to the bus. The bus rolled onto the railing and came to rest on its side after leaving the railing. This tendency to roll may have been aggravated by the fact that the curb did not deflect laterally while the metal rail element did. Such action would allow the upper portion of the bus to lean over while the bottom of the wheels were being restrained by the non-deflecting curb.

Performance of this railing design is considered inadequate even for the automobile portion of the spectrum of vehicles.



COLORADO TYPE 5 BRIDGE RAIL

Test No. 3451-1
 Date 1/04/78
 Rail TS8x6x0.25
 Post TS5x5x0.05
 Post Spacing 2.35 m (7.71 ft)
 Length of Installation 45.75 m (150 ft)
 Beam Deflection
 Max. Dynamic 0.03 m (0.10 ft)
 Max. Permanent 0.00 m (0.00 ft)
 Vehicle Damage
 TAD 1RFQ4
 SAE 01RFEW2

Vehicle 1974 Vega
 Vehicle Weight 1256 kg (2770 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 90.1 kph (56.0 mph)
 Impact Angle 15.1 deg
 Exit Speed 68.7 kph (42.7 mph)
 Exit Angle 4.2 deg
 Vehicle Acceleration
 (Max. 0.050 sec)
 Longitudinal -5.8 g
 Transverse 7.2 g
 Vertical -3.1 g

Figure 14. Summary of Results for Test 3451-1.

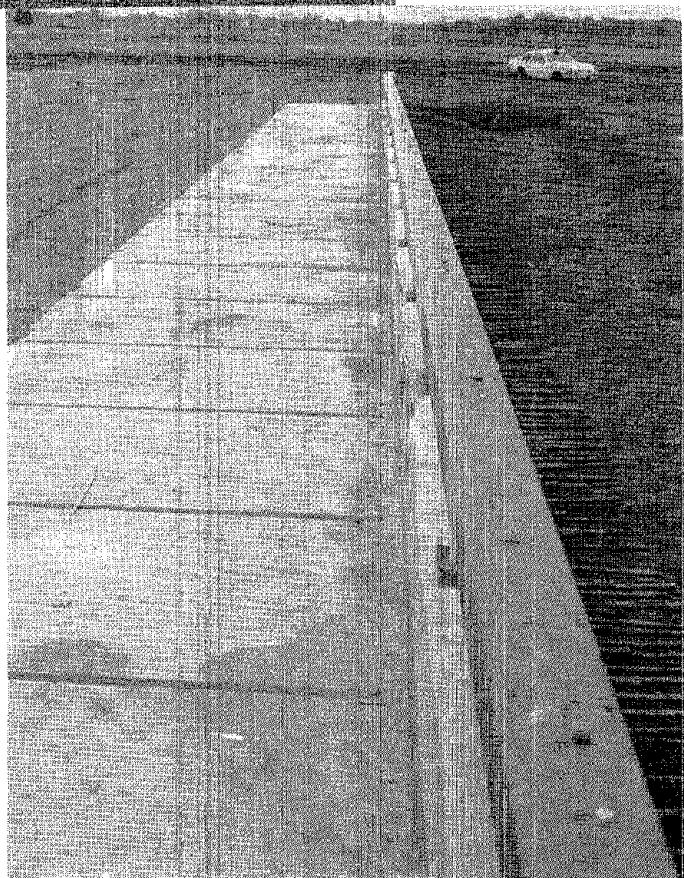
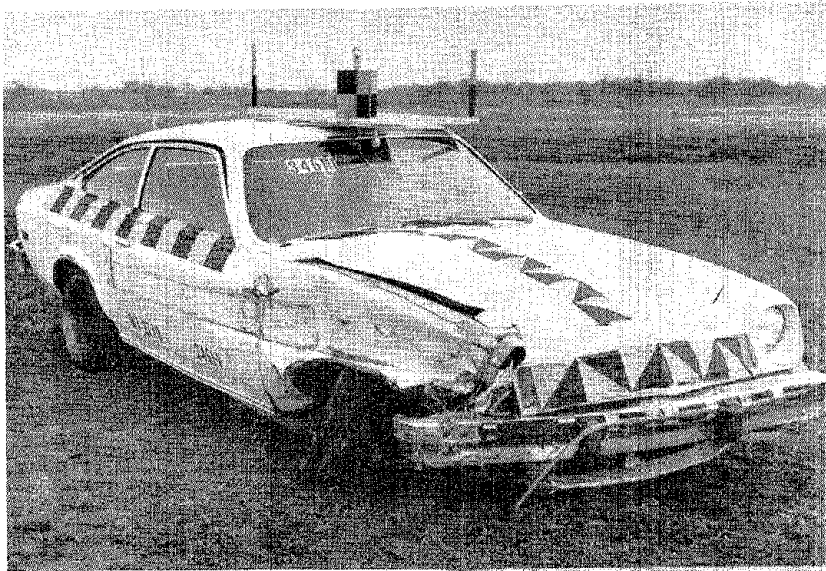
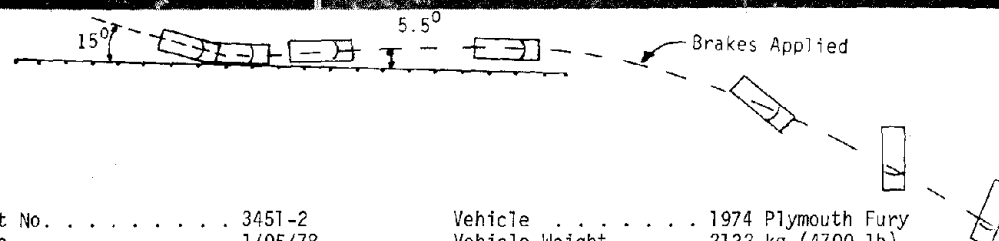
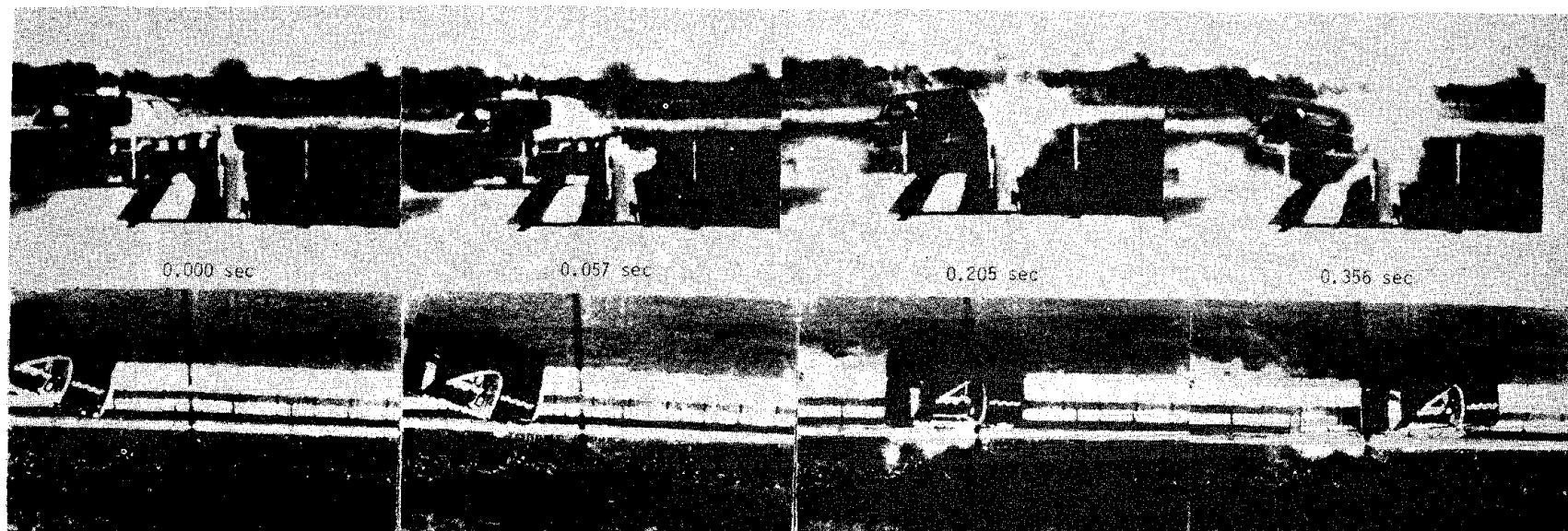
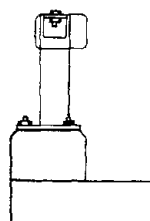


Figure 15. Photographs of Vehicle and Railing After Test 3451-1.



Test No.	3451-2	Vehicle	1974 Plymouth Fury
Date	1/05/78	Vehicle Weight	2132 kg (4700 lb)
Rail	TS8x6x0.25	(w/dummies, instr.)	
Post	TS5x5x0.05	Dummy Restraints	
Post Spacing	2.35 m (7.71 ft)	Driver	Lap Belt
Length of Installation	45.75 m (150 ft)	Passenger	No Restraint
Beam Deflection		Impact Speed	101.1 kph (62.8 mph)
Max. Dynamic	0.12 m (0.40 ft)	Impact Angle	15.0 deg
Max. Permanent	0.06 m (0.20 ft)	Exit Speed	75.3 kph (46.8 mph)
Vehicle Damage		Exit Angle	5.5 deg
TAD	1RFQ4	Vehicle Acceleration	
SAE	01RFEE2	(Max. 0.050 sec)	
		Longitudinal	-4.8 g
		Transverse	5.8 g
		Vertical	1.6 g



COLORADO TYPE 5 BRIDGE RAIL

Figure 16. Summary of Results for Test 3451-2.

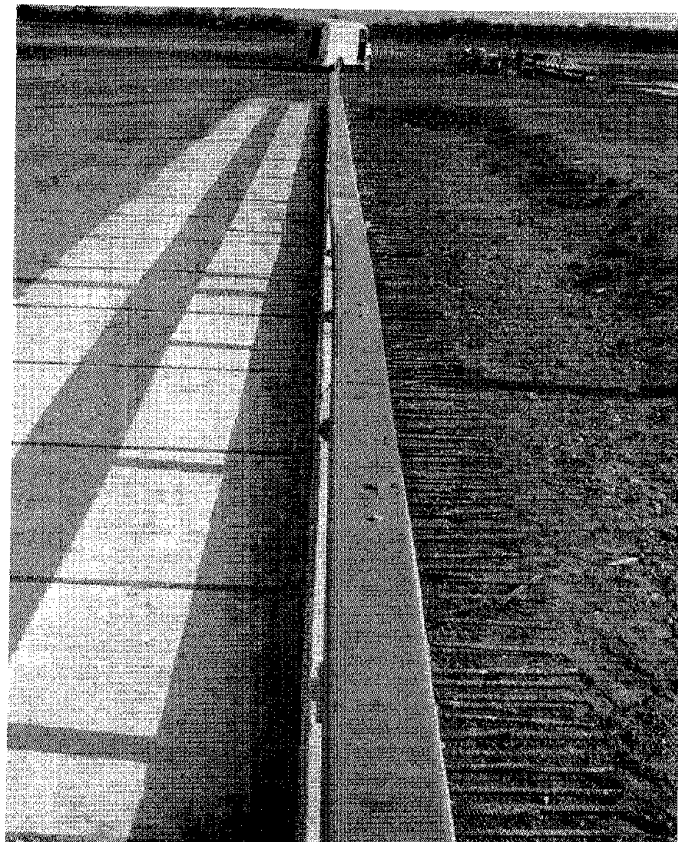
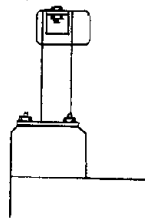
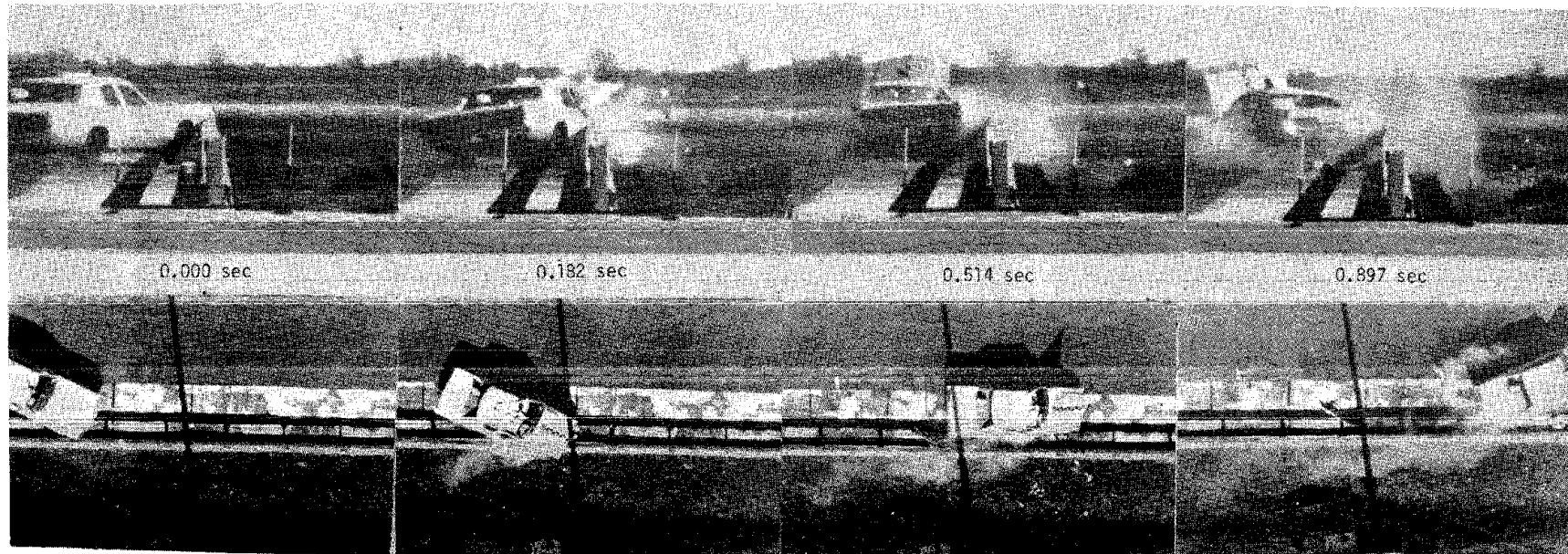


Figure 17. Photographs of Vehicle and Railing After Test 3451-2.



COLORADO TYPE 5 BRIDGE RAIL

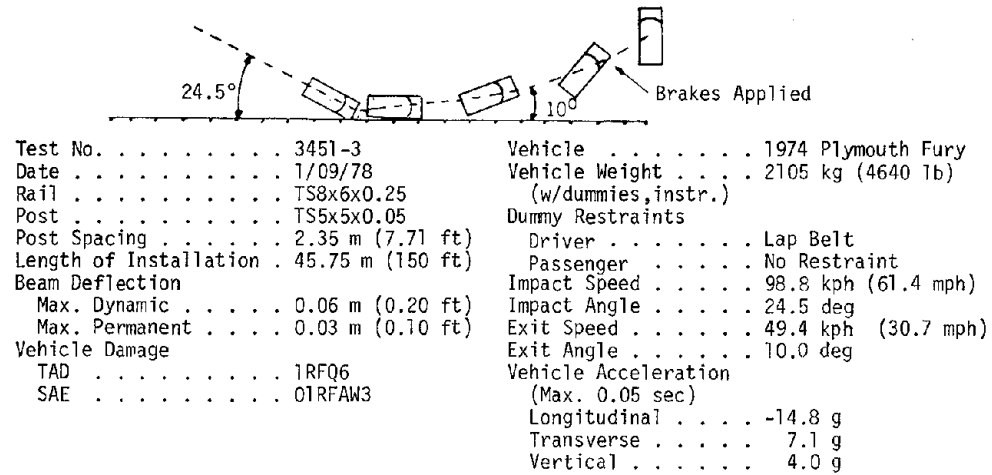


Figure 18. Summary of Results for Test 3451-3.

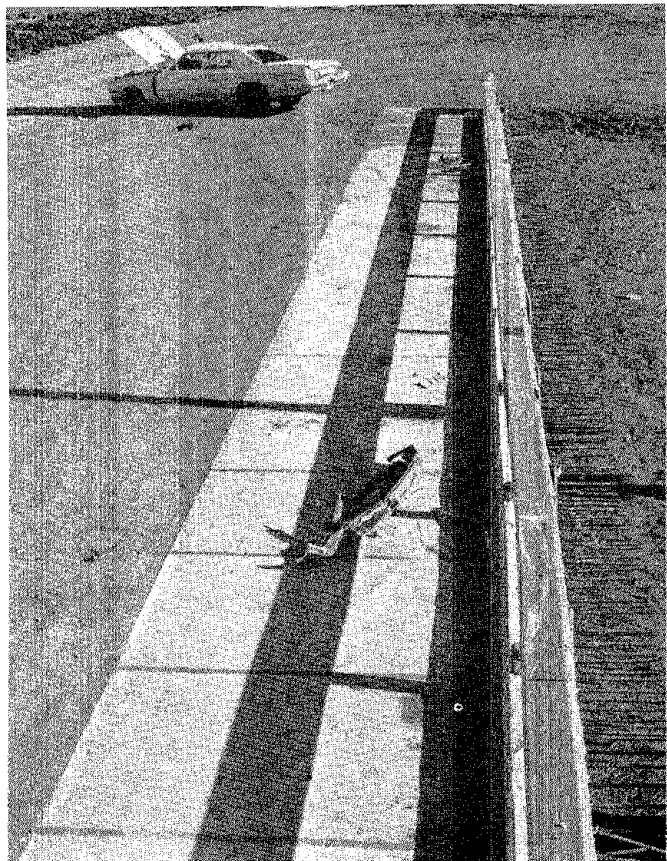
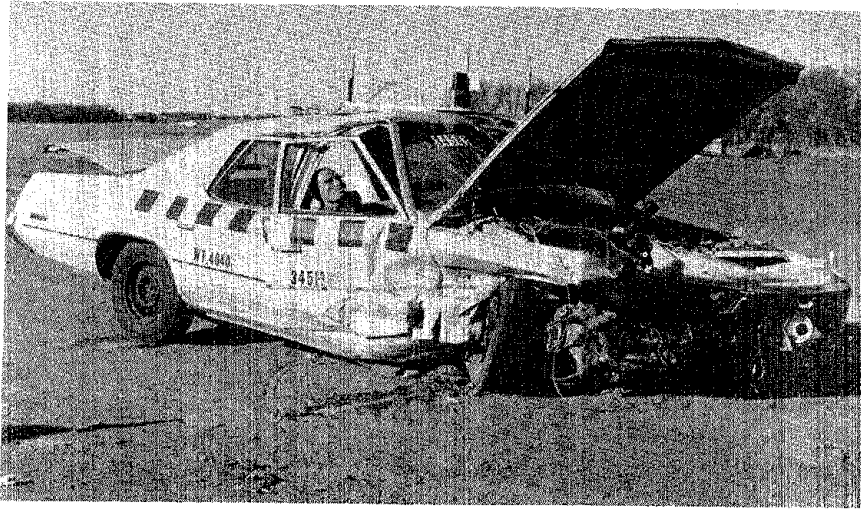
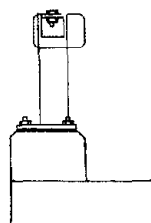
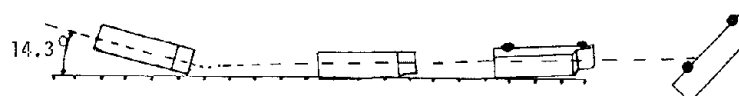
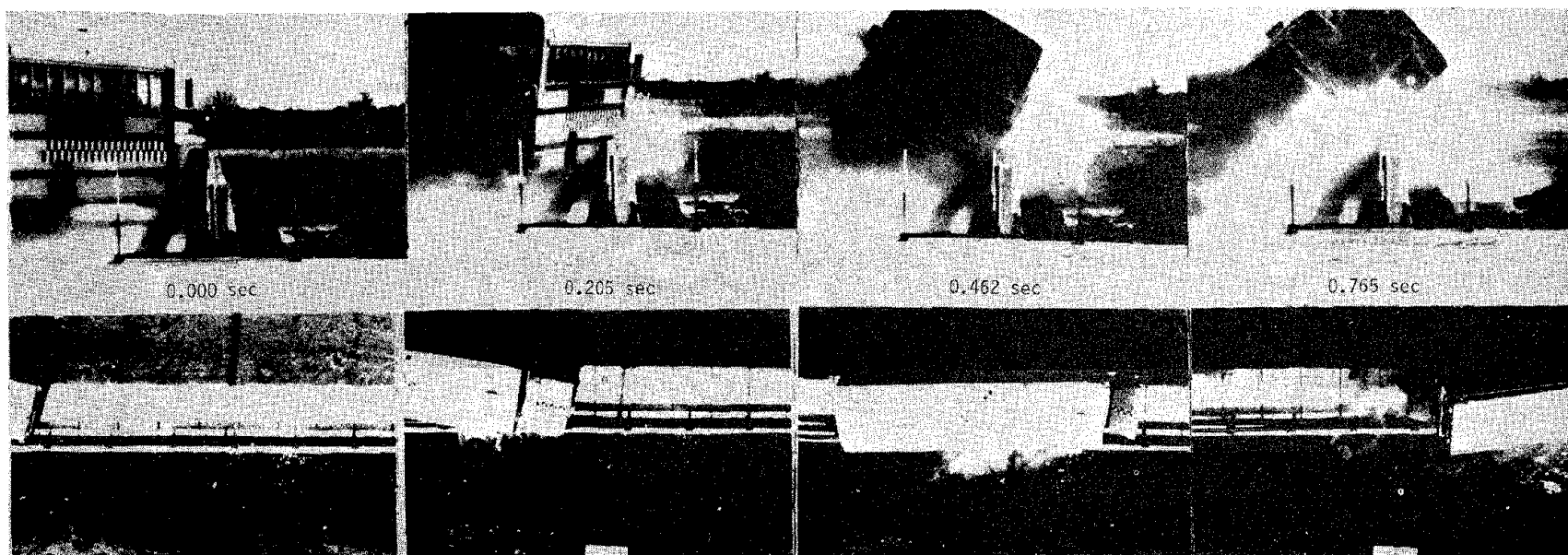


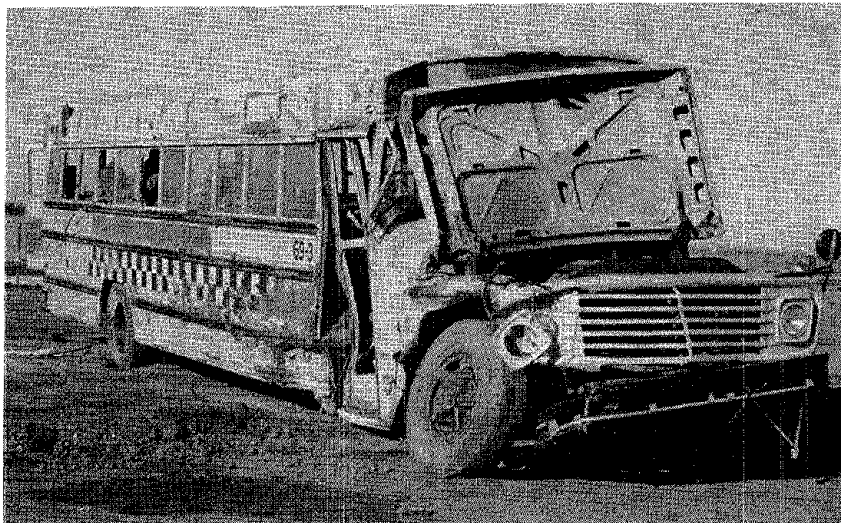
Figure 19. Photographs of Vehicle and Railing After Test 3451-3.



COLORADO TYPE 5 BRIDGE RAIL

Test No.	3451-4	Vehicle	1969 Ford School Bus
Date	2/23/78	Vehicle Weight	8963 kg (19,760 lb)
Rail	TS8x6x0.25	(w/dummies,instr.)	
Post	TS5x5x0.05	Dummy Restraints	
Post Spacing	2.35 m (7.71 ft)	Driver	Lap Belt
Length of Installation	45.75 m (150 ft)	Passengers	No Restraint
Beam Deflection		Impact Speed	95.6 kph (59.4 mph)
Max. Dynamic	0.12 m (0.40 ft)	Impact Angle	14.3 deg
Max. Permanent	0.06 m (0.20 ft)	Exit Speed	No data available-
Vehicle Damage		Exit Angle	Bus rolled
TAD	1RFQ6	Vehicle Acceleration	
SAE	01RFAW3	(Max. 0.050 sec)	
Maximum Roll	Vehicle rolled onto side	Longitudinal	-1.6 g
		Transverse	4.7 g
		Vertical	-1.8 g

Figure 20. Summary of Results for Test 3451-4.



Bus after
being
uprighted

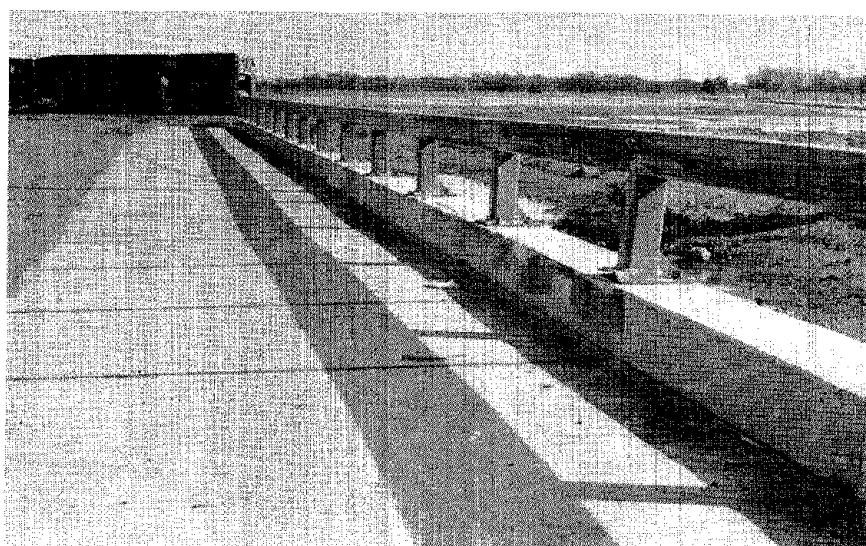


Figure 21. Photographs of Vehicle and Railing After
Test 3451-4.

Results for Texas T101 Railing: A series of seven tests (Numbers 5 through 11) was conducted on the Texas T101 railing. No Honda [817 kg (1,800 lb)] test vehicles were included in this series. Very clean, smooth redirections occurred in all tests with automobiles. In automobile tests [1,022 and 2,043 kg (2,250 and 4,500 lb) vehicle] with 15 deg impact angles, the highest 0.050 sec average lateral vehicle accelerations exceeded the 5.0 g limit specified in TRC 191 (3). In the Vega [1,022 kg (2,250 lb)] test, this acceleration was 7.3 g's. Subjectively, performance of the railing system, in automobile tests, would be considered to be very good. The acceleration levels obtained are probably as good as can be expected from a railing design that does not include some type of energy absorbing feature or which is not designed to deflect significantly during automobile impacts. Tests on other railing systems have produced similar results. This leads one to question the appropriateness of the 5.0 g limit specified in TRC 191 (3). If that requirement were strictly followed, most, if not all, railing systems would be required to provide significant energy absorption in order to meet performance requirements in automobile tests.

In the bus tests, the vehicles were contained and redirected; however, the 14,528 kg (32,000 lb) intercity bus almost penetrated the railing. Maximum deflection of the railing was almost 1.5 m (5 ft.) The bus rolled onto the railing and came to rest on its side (rolled 90 deg) after leaving the end of the railing. A 9,080 kg (20,000 lb) school bus was easily contained and redirected by the railing with acceptable roll stability of the vehicle. However, in this test, a somewhat unique combination of vehicle and railing characteristics may have resulted in successful performance. During the initial portion of the collision, most of the force was transmitted through the front axle assembly causing the entire assembly to be separated from the vehicle. This behavior is argued to have resulted in low collision forces and a lowering of the center of gravity height thereby providing roll stability to the vehicle. Use of a different make of vehicle with a different suspension system and/or a slightly taller railing may have resulted in rollover of the vehicle.

Qualitatively, a performance of this railing is considered adequate

for the 1,022 to 2,043 kg (2,250 to 4,500 lb) automobile spectrum although the lateral acceleration limit specified in TRC 191 (3) was not met.

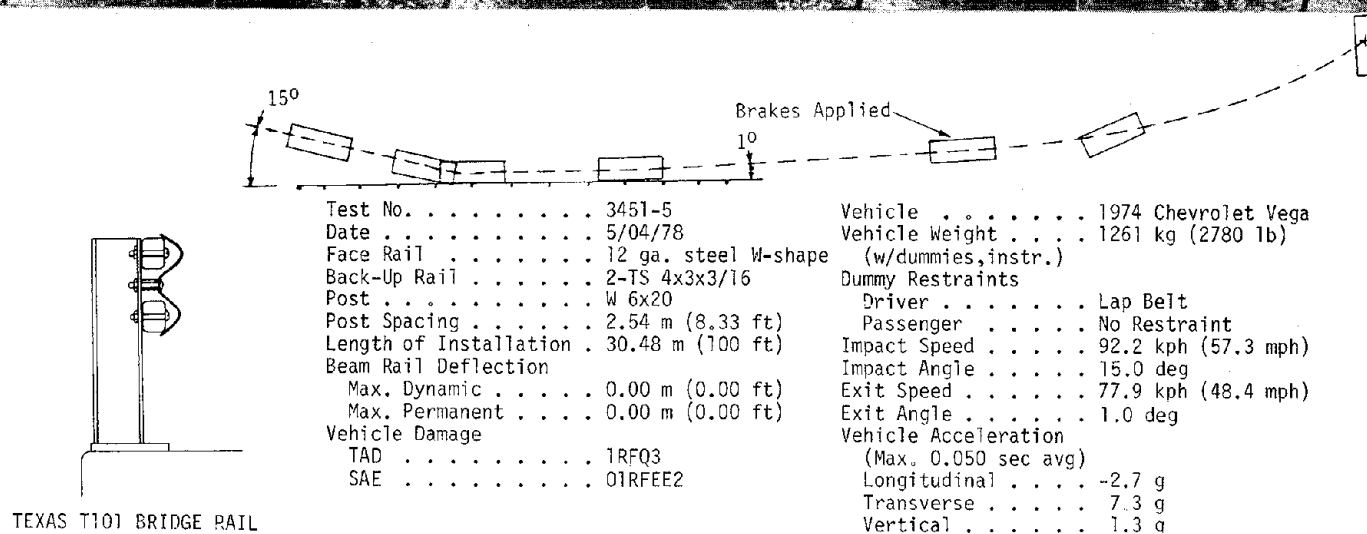
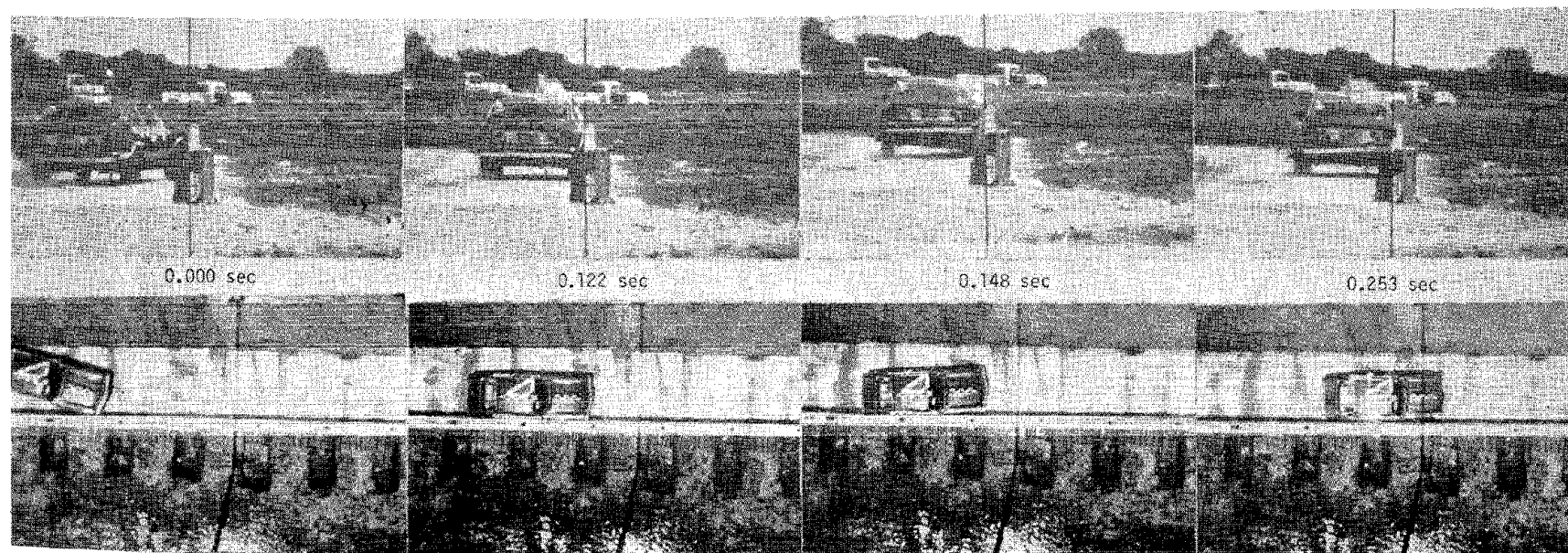


Figure 22 . Summary of Results for Test 3451-5.

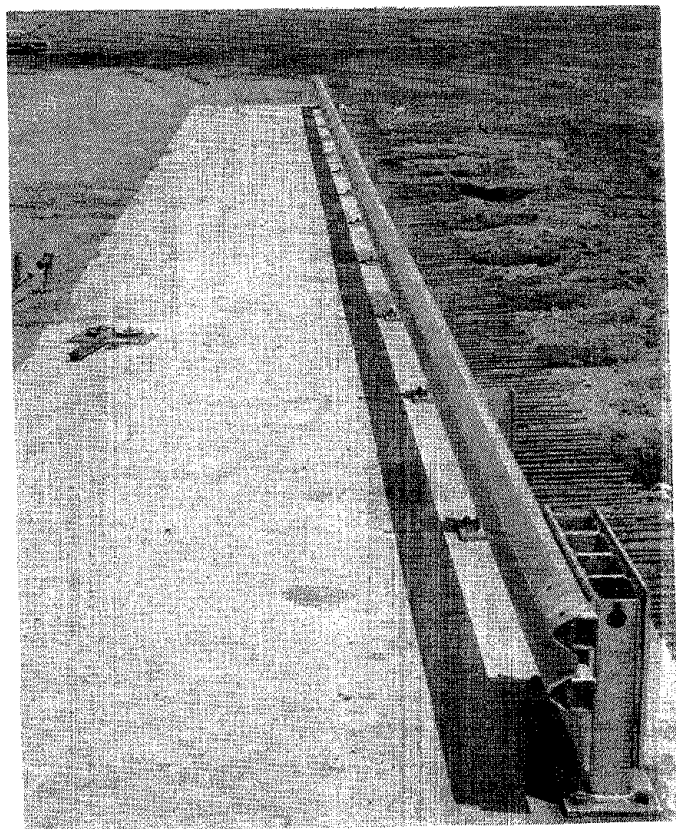


Figure 23. Photographs of Vehicle and Railing After Test 3451-5.

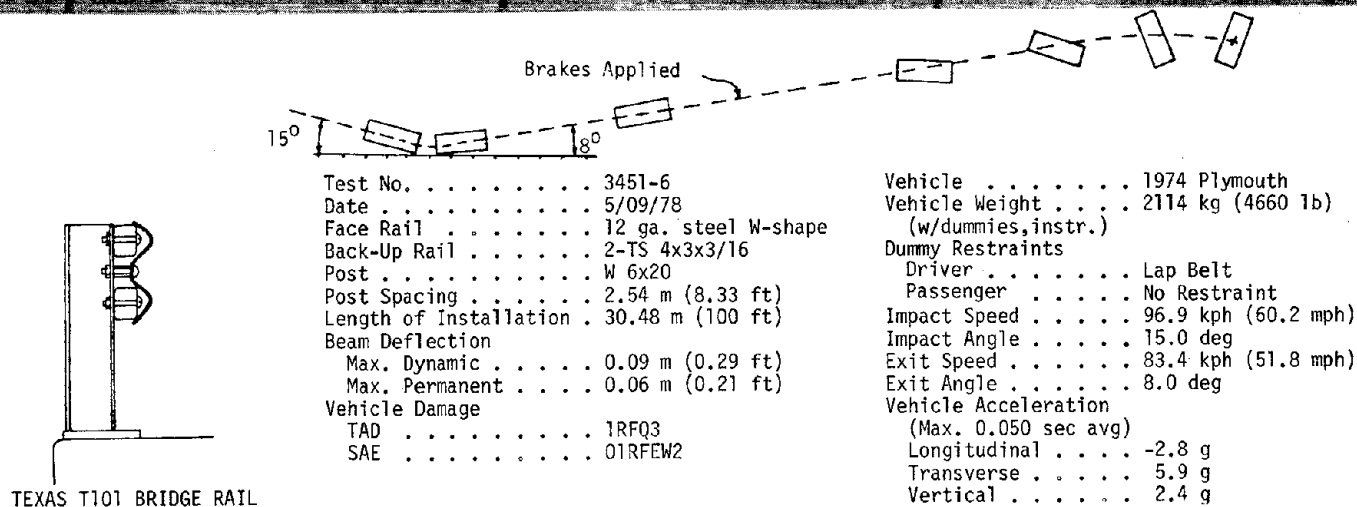
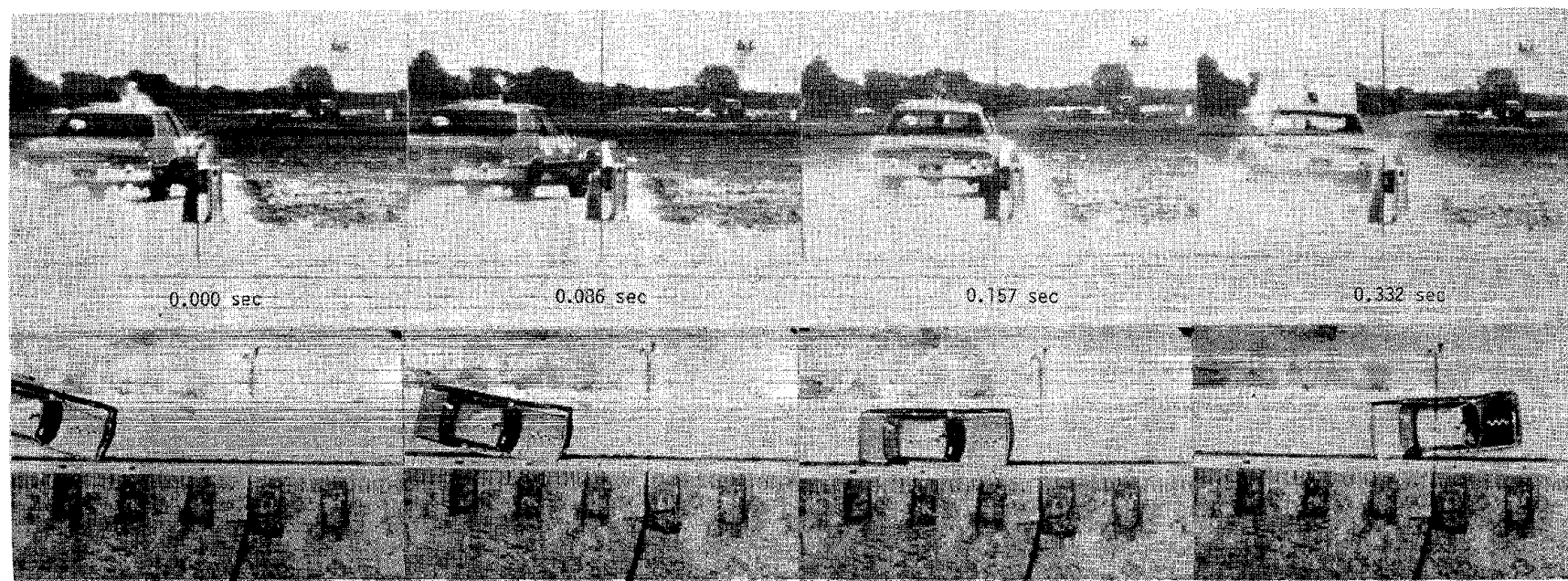


Figure 24. Summary of Results for Test 3451-6.

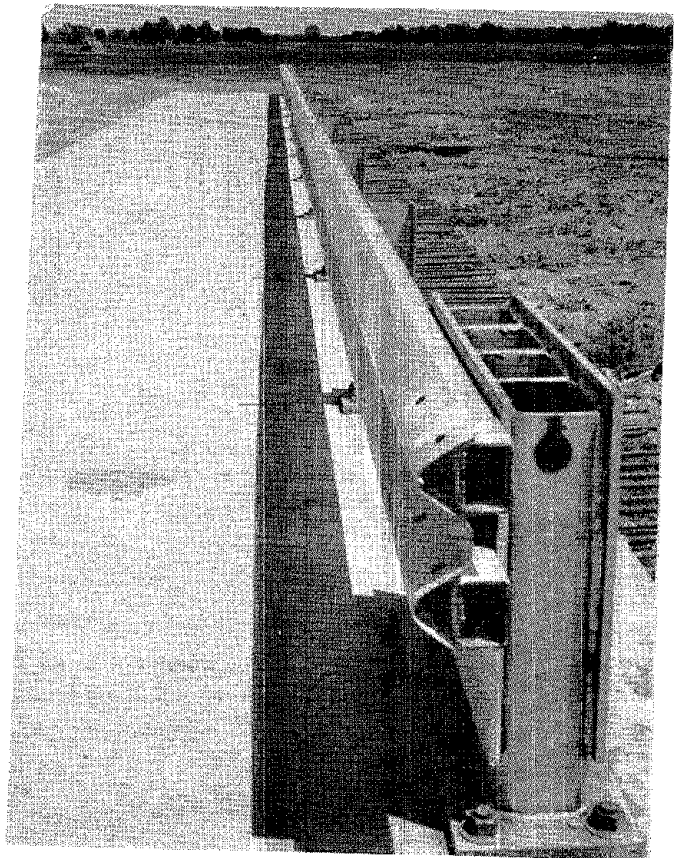
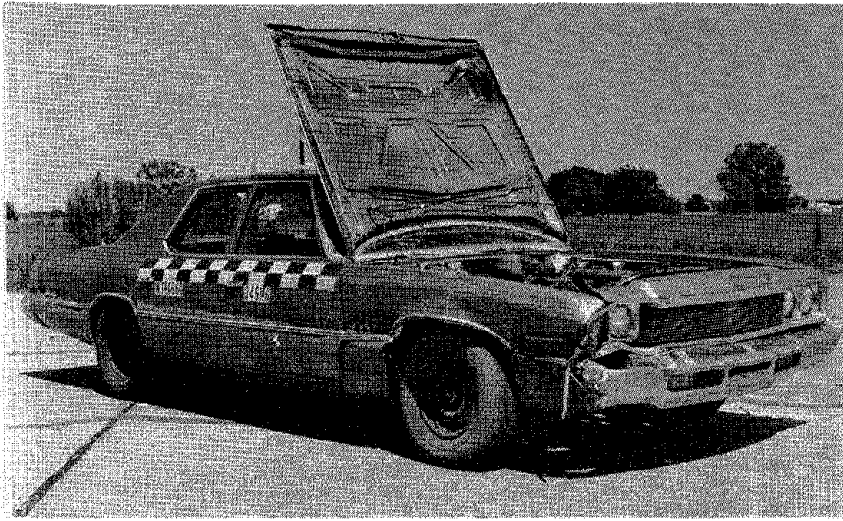


Figure 25. Photographs of Vehicle and Railing After Test 3451-6.

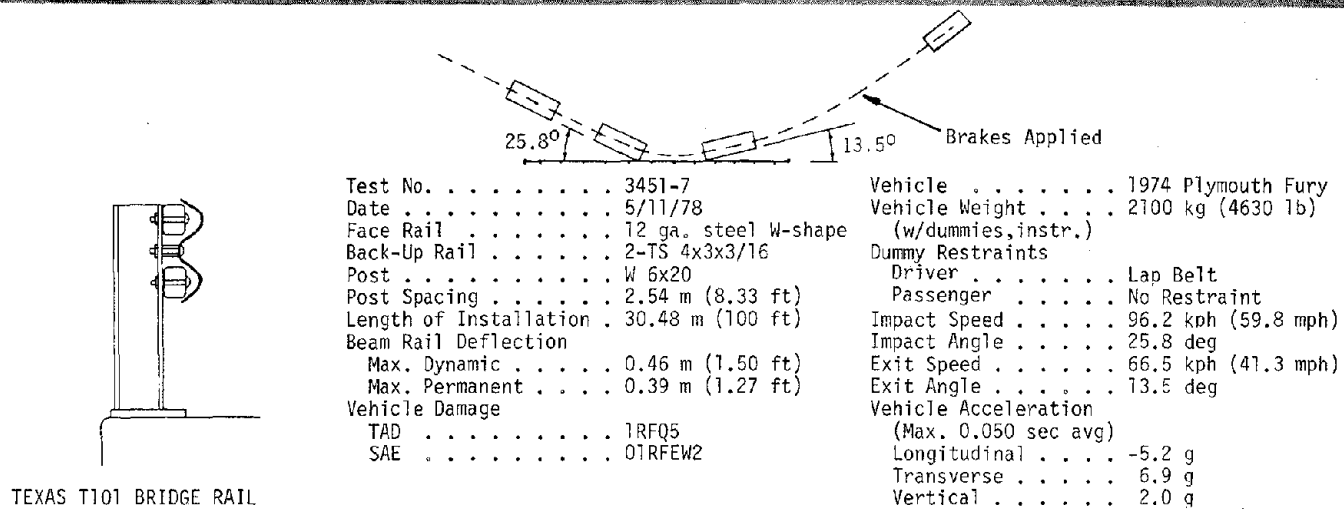
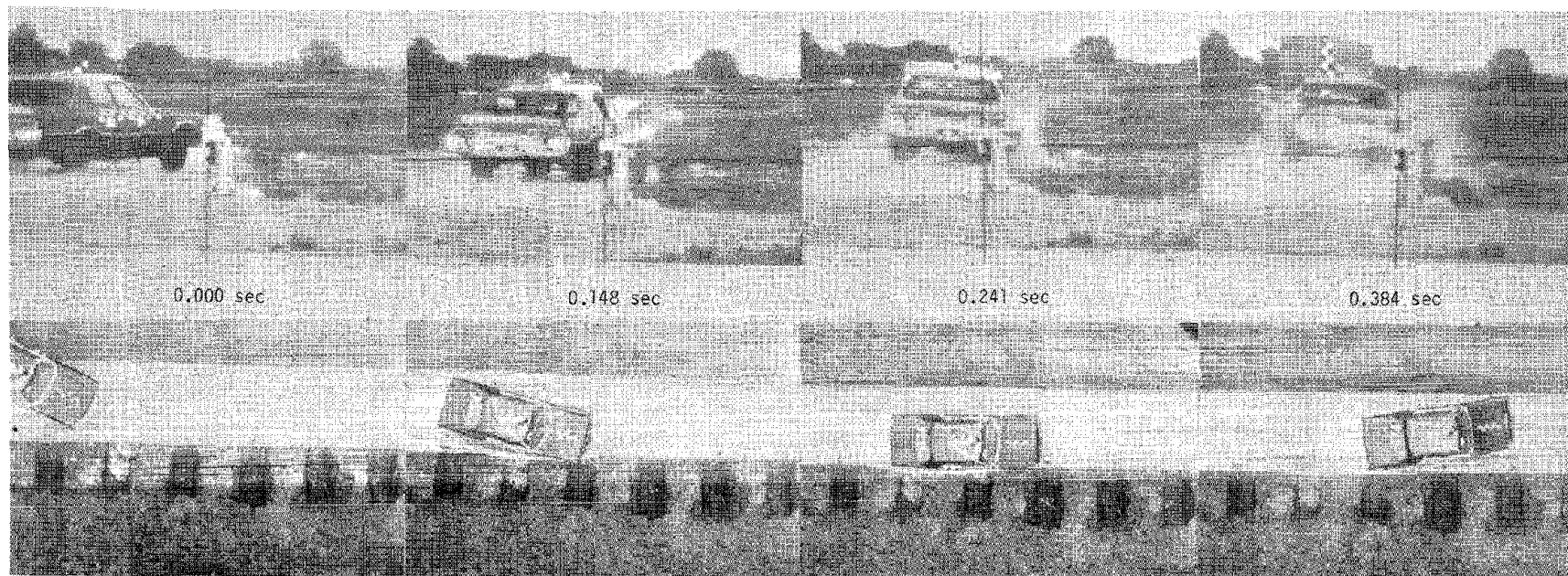


Figure 26. Summary of Results for Test 3451-7.

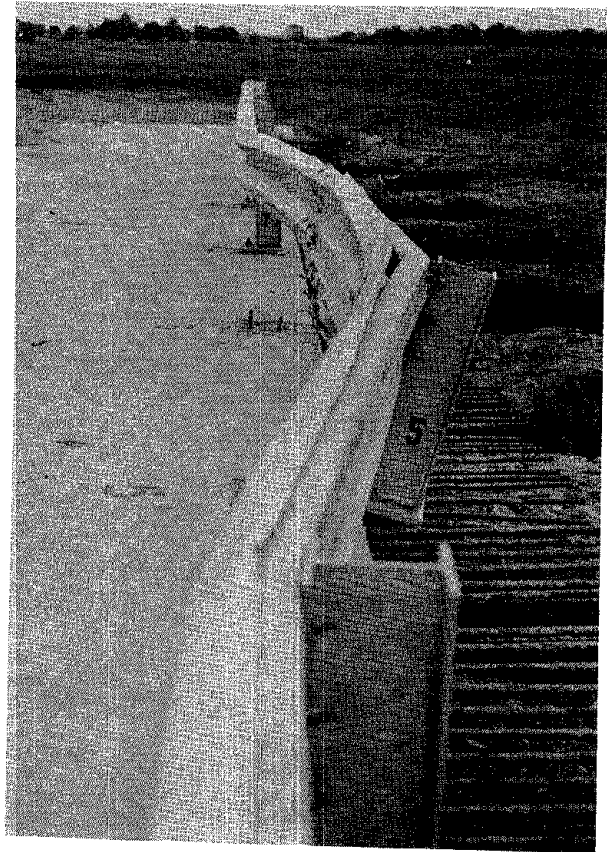
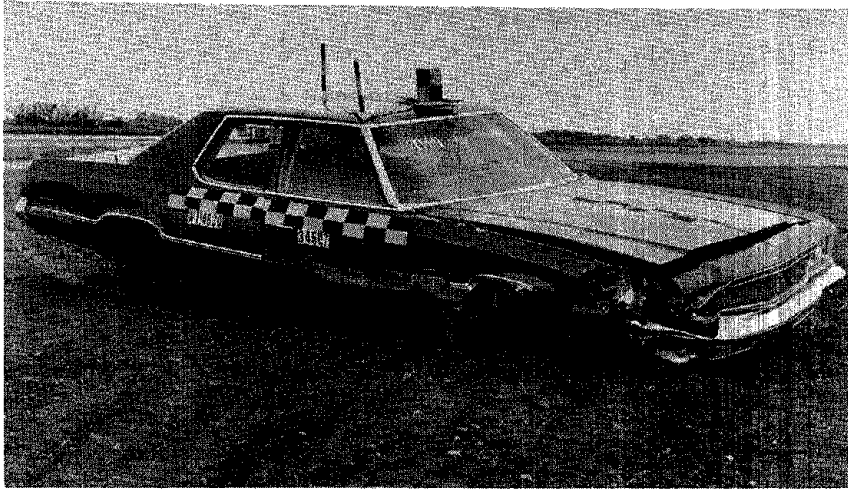


Figure 27. Photographs of Vehicle and Railing After Test 3451-7.

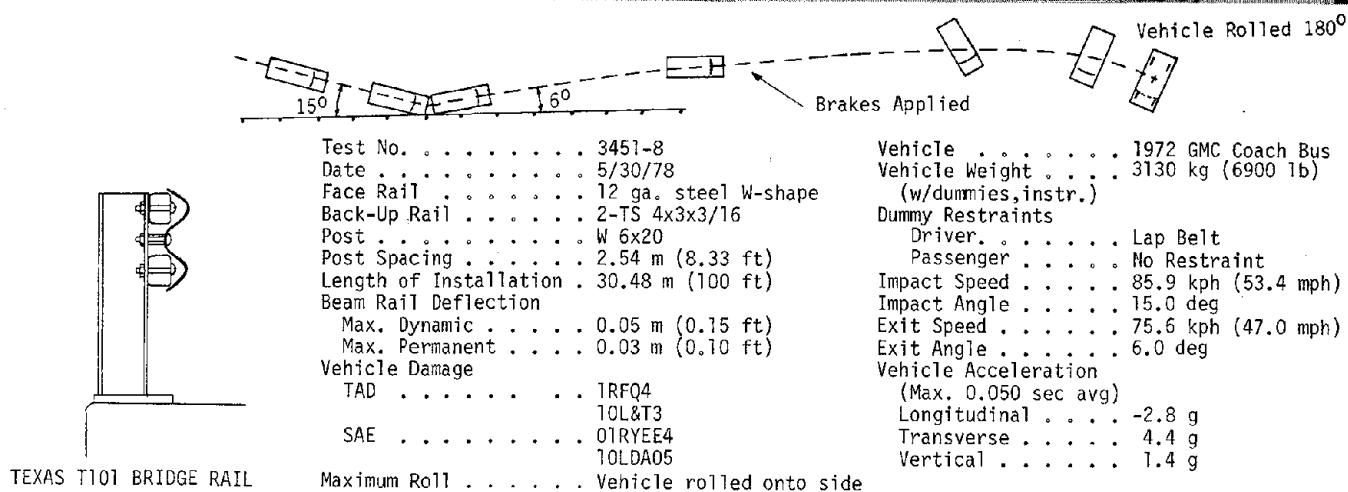
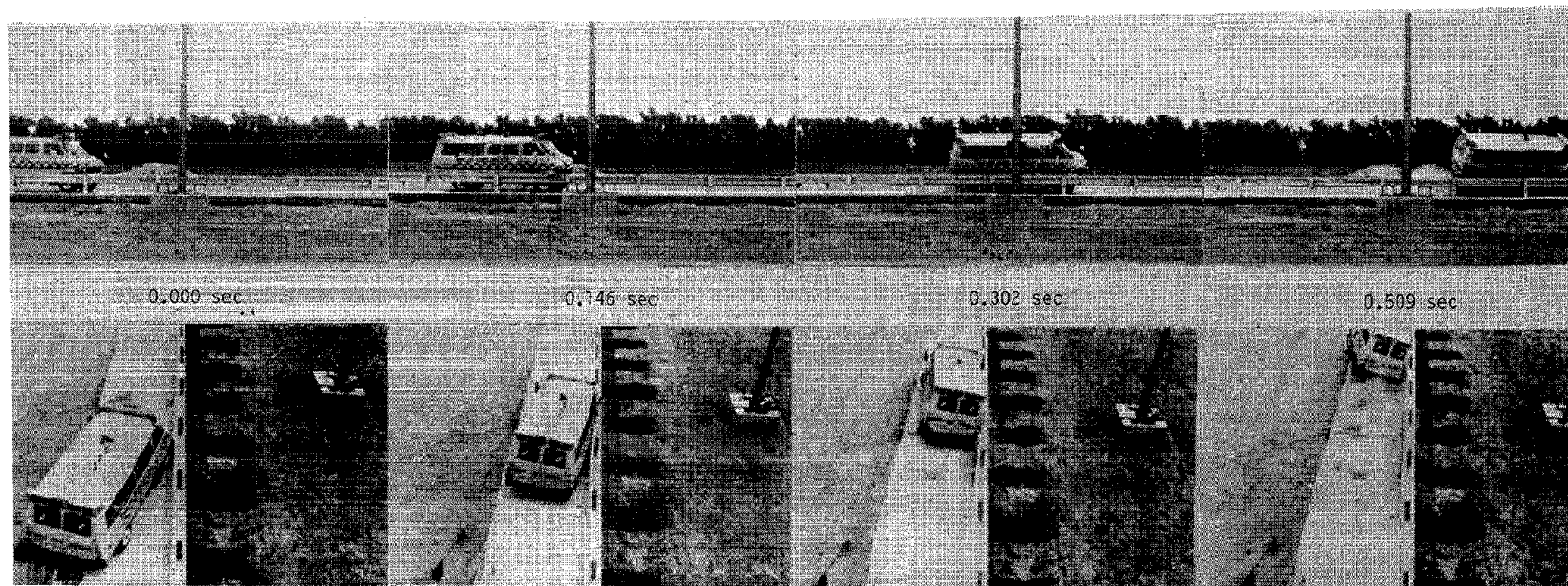


Figure 28. Summary of Results for Test 3451-8.

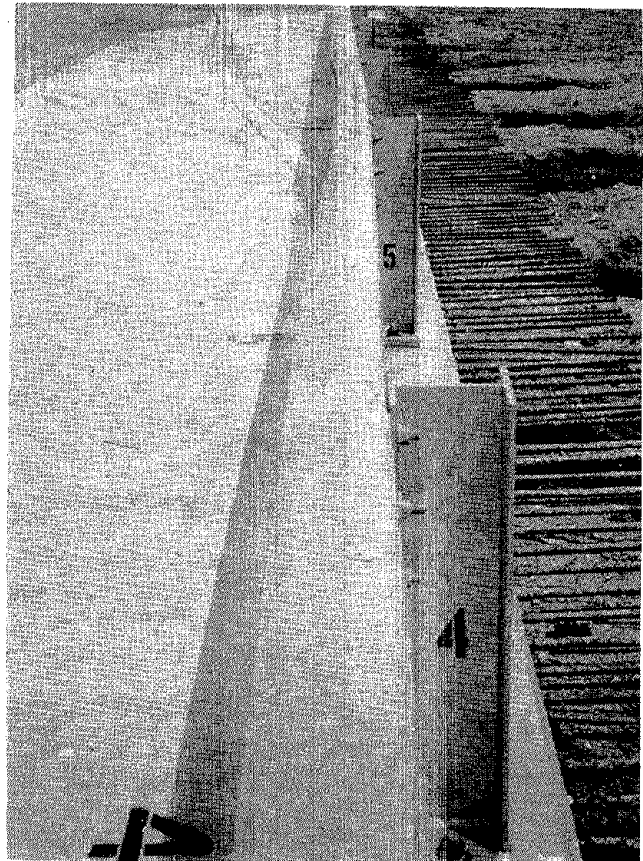
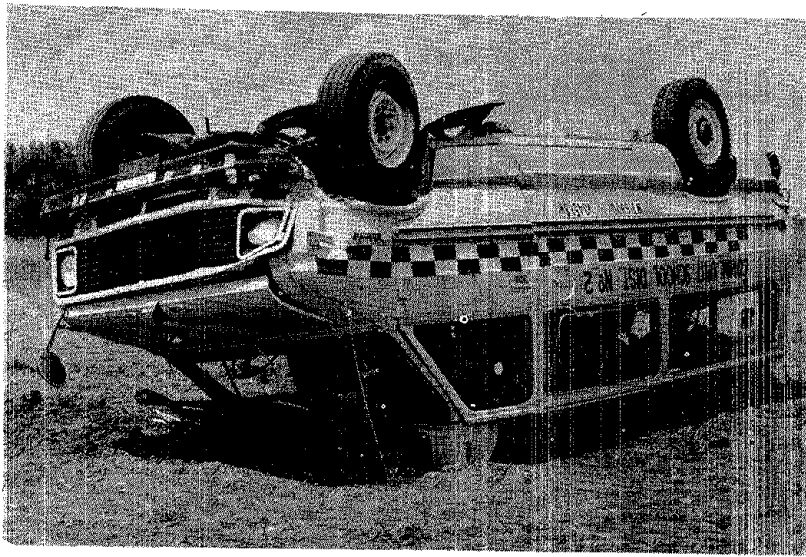
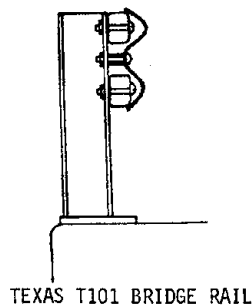
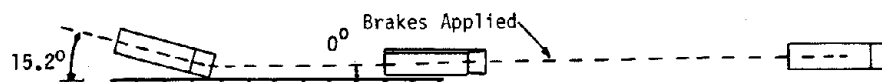
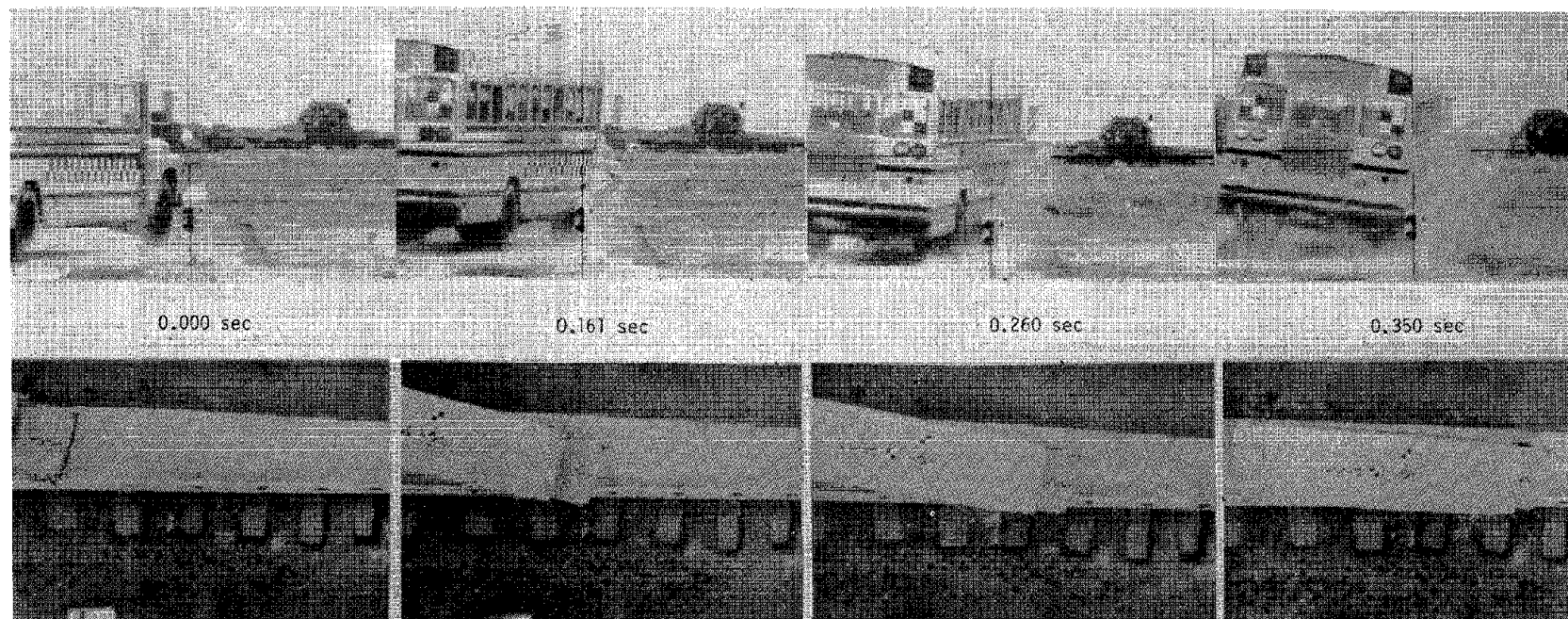


Figure 29. Photographs of Vehicle and Railing After Test 3451-8.



Test No. 3451-9
 Date 6/08/79
 Face Rail 12 ga. steel W-shape
 Back-Up Rail 2-TS 4x3x3/16
 Post W 6x20
 Post Spacing 2.54 m (8.33 ft)
 Length of Installation 30.48 m (100 ft)
 Beam Rail Deflection
 Max. Dynamic 0.23 m (0.77 ft)
 Max. Permanent 0.08 m (0.26 ft)
 Vehicle Damage
 TAD 1RFQ5
 SAE 01RFEE2
 Maximum Roll 36.0 deg

Vehicle 1969 Ford Bus
 Vehicle Weight 9045 kg (19,940 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passengers No Restraints
 Impact Speed 89.0 kph (55.3 mph)
 Impact Angle 15.2 deg
 Exit Speed No Data
 Exit Angle 0 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -1.6 g
 Transverse 4.2 g
 Vertical -1.2 g

Figure 30. Summary of Results for Test 3451-9.

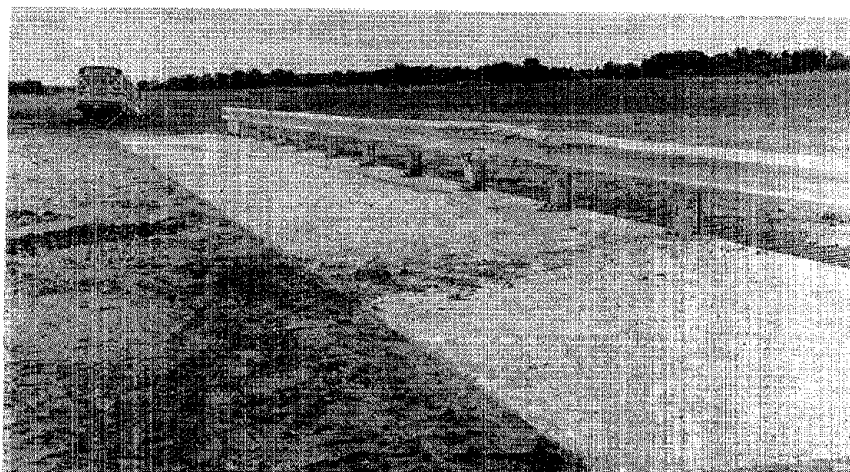
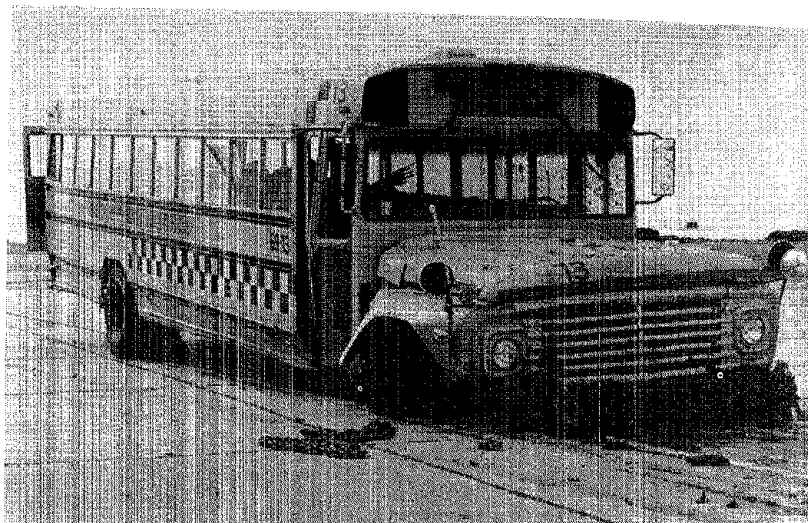


Figure 31 . Photographs of Vehicle and Railing After
Test 3451-9.

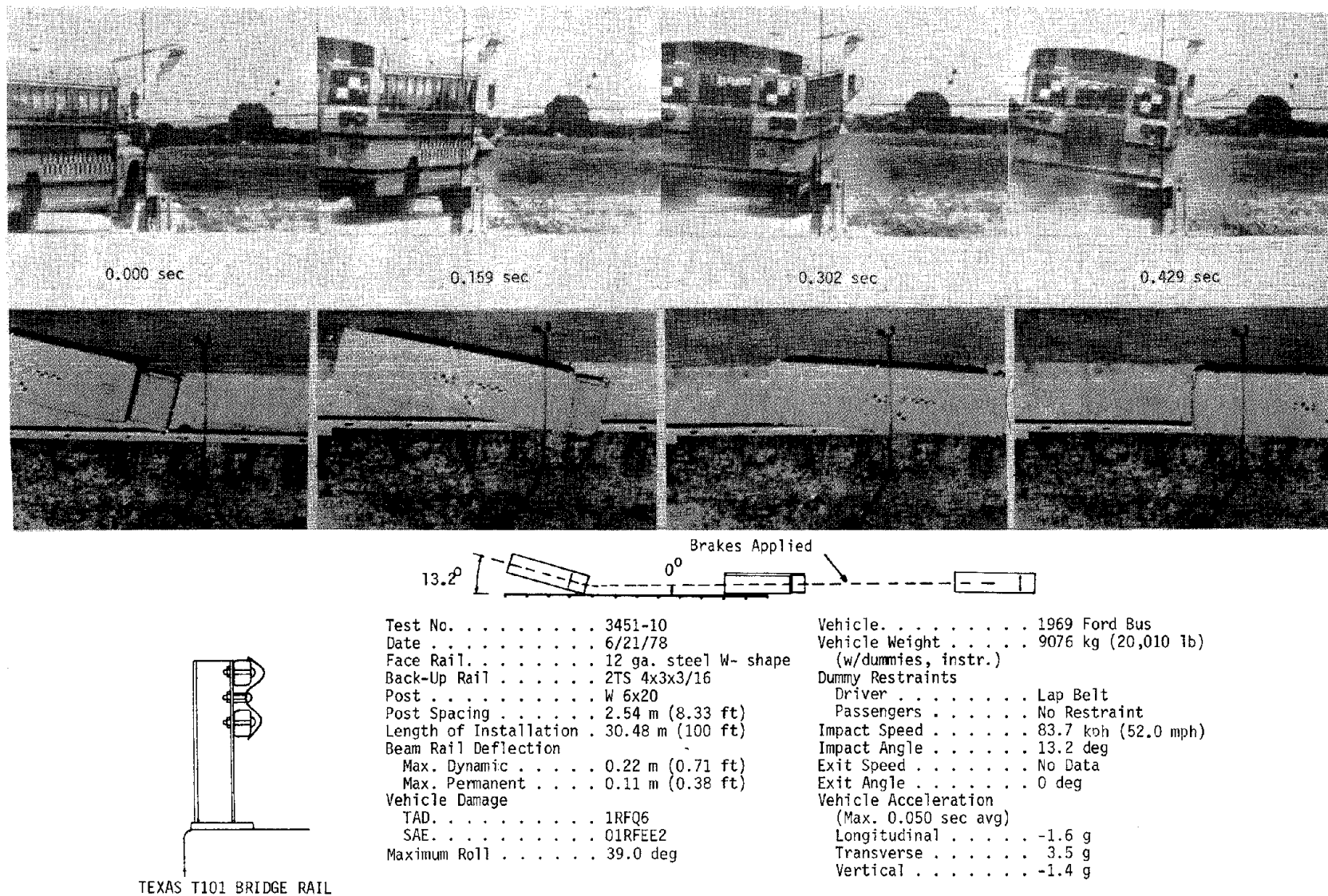


Figure 32. Summary of Results for Test 3451-10.

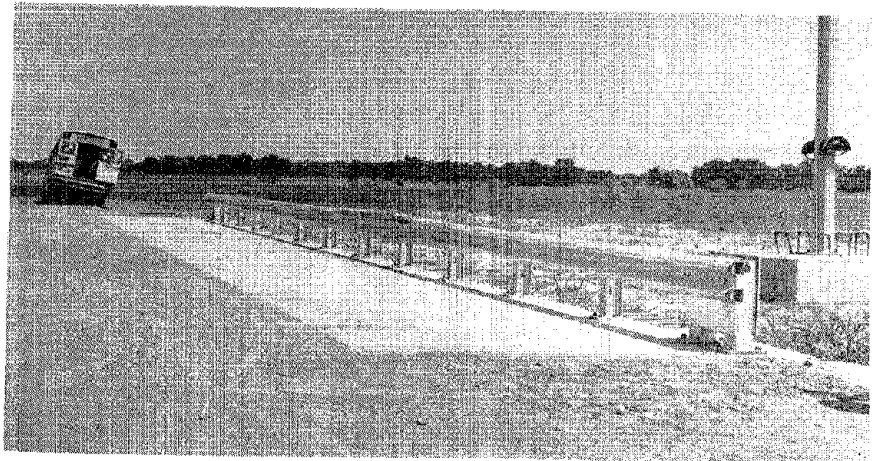
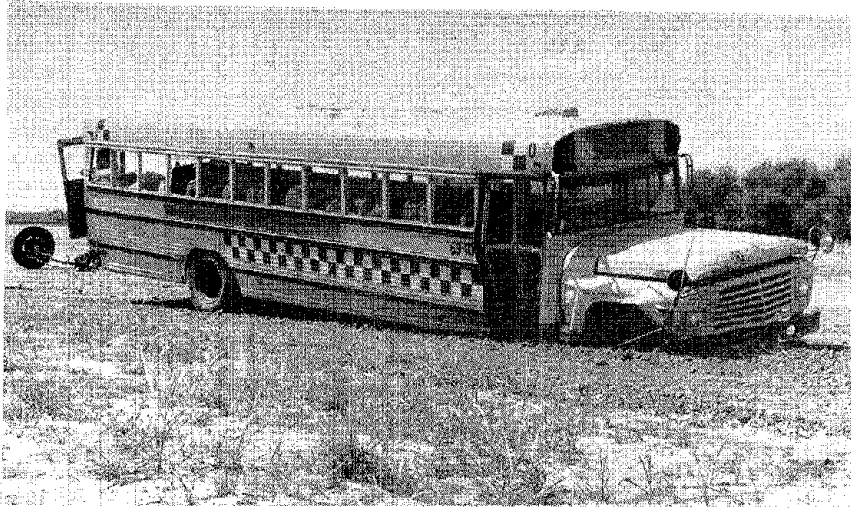


Figure 33. Photographs of Vehicle and Railing After Test 3451-10.

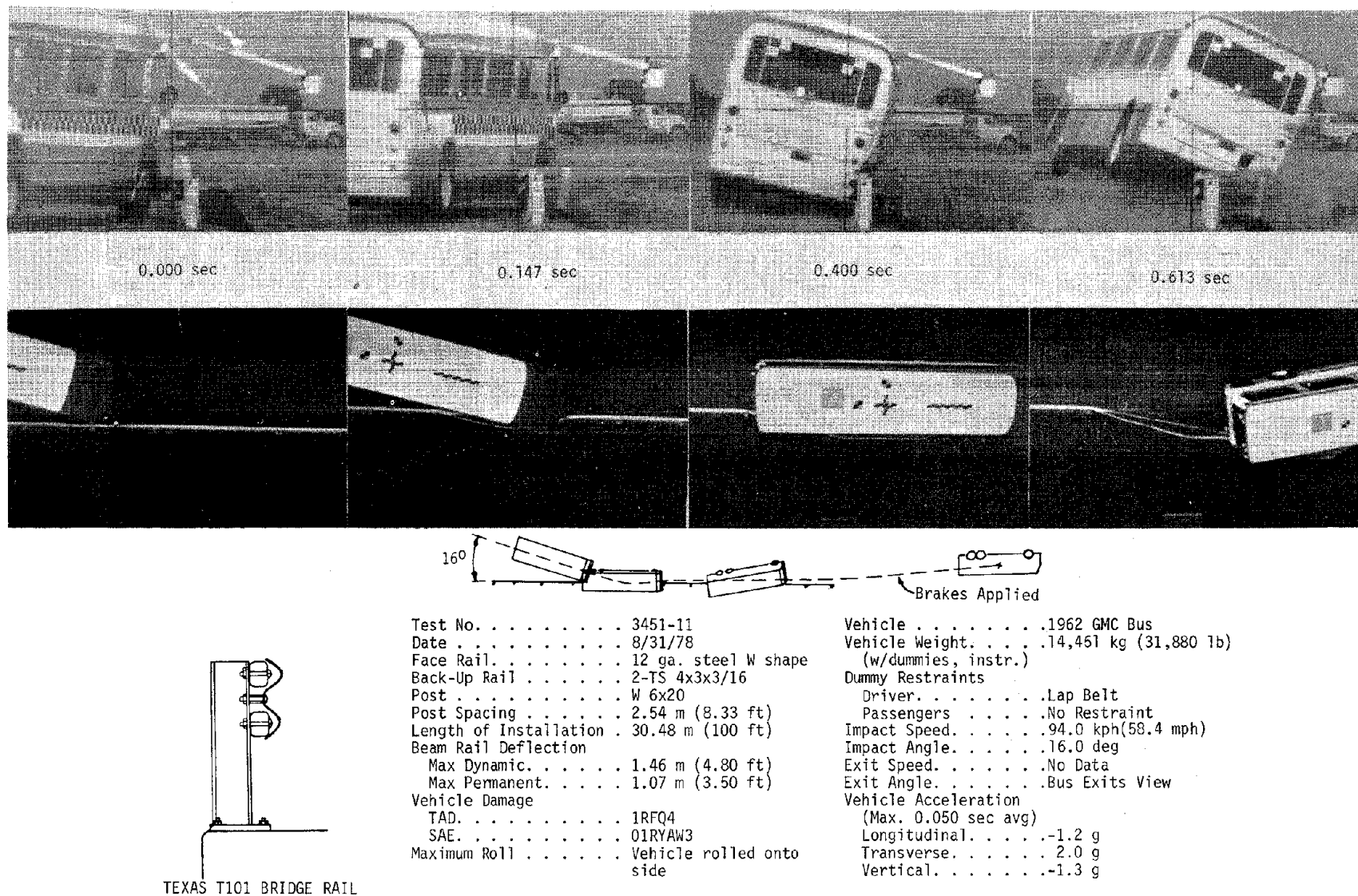


Figure 34. Summary of Results for Test 3451-11.

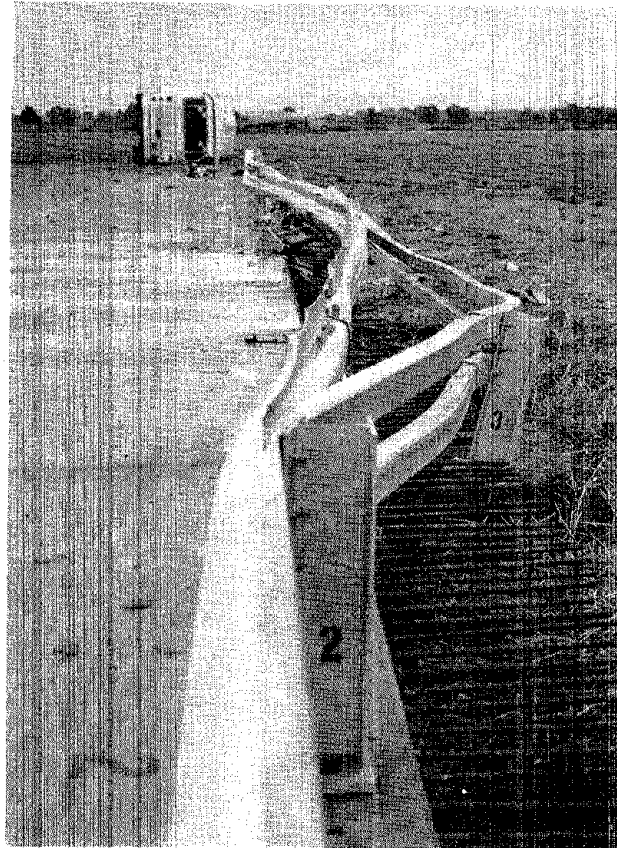
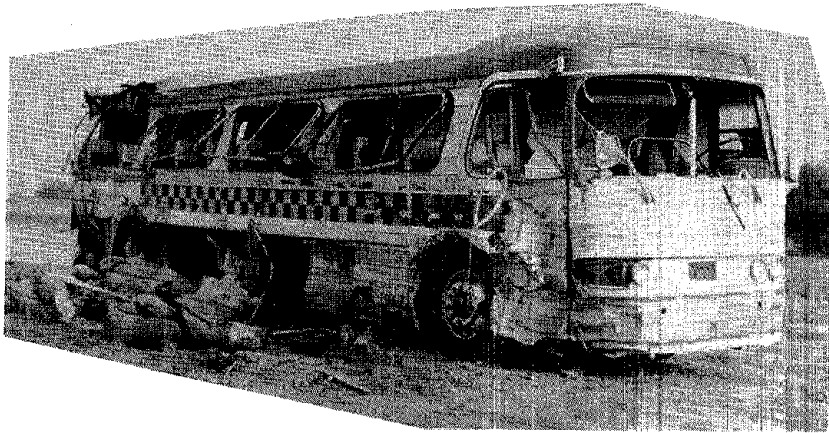
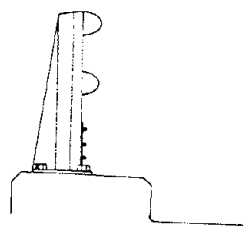
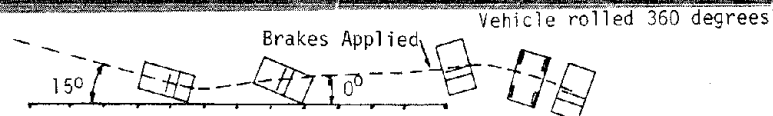
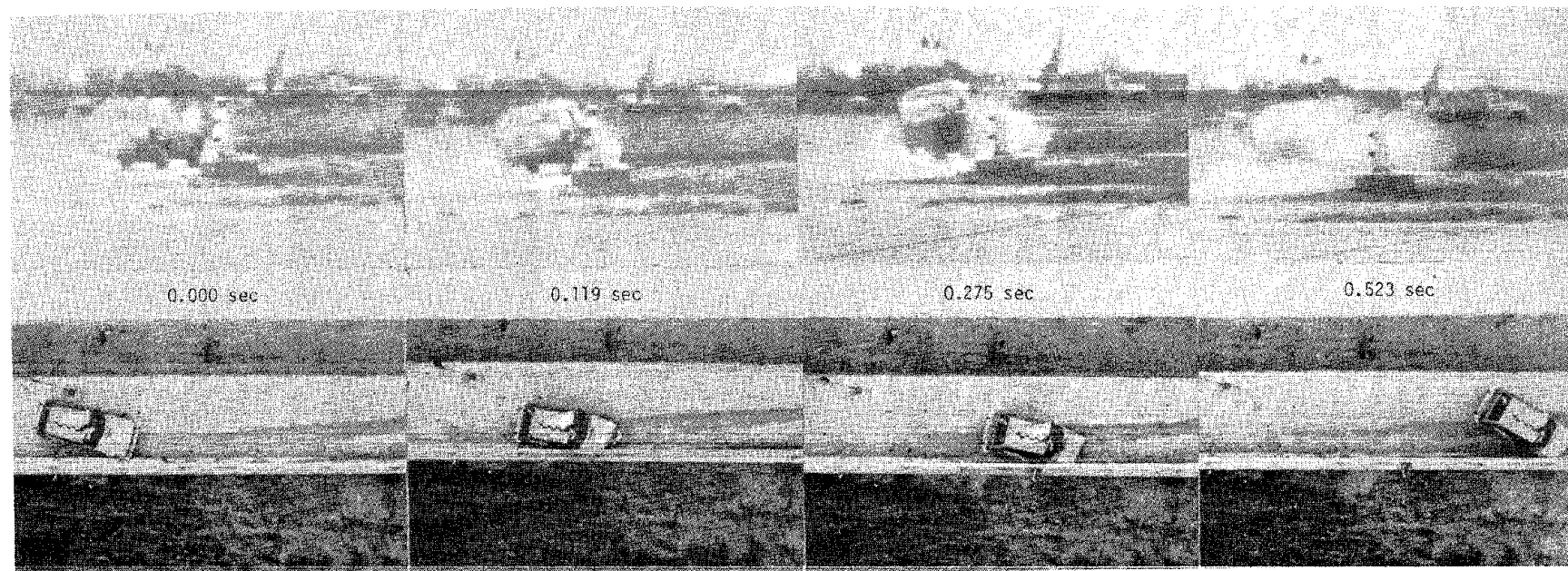


Figure 35. Photographs of Vehicle and Railing After Test 3451-11.

Results for New Hampshire Railing: A series of four tests (Numbers 12 through 15) was conducted on the New Hampshire railing system. This railing design has undesirable geometrics and failed to produce smooth redirections in automobile tests. The presence of the protruding curb coupled with the vertical location of the open space caused severe snagging in tests with automobiles. The railing system includes an 0.22 m (8 1/2 in.) high curb (which projects 0.23 m (9 in.) from the traffic face of the metal railing) with a 0.36 m (14 in.) vertical opening immediately above the curb. As the automobile approached the railing, the front wheel encountered the curb and was damaged. Some uplift of the wheel occurred but the wheel essentially continued forward with the metal rim rolling on top of the curb. This phenomenon was more pronounced with the smaller automobiles. The wheel in this position was then most disadvantageously aligned with the open space in the metal railing immediately above the curb. The wheel, especially on the smaller automobiles, then continued forward to penetrate the open space and snag on railing posts.

The post baseplate also has a structural deficiency that was overshadowed by the influence of the railing geometrics but was identified in later testing of the Indiana 5A railing which uses a similar post and baseplate.

Performance of the New Hampshire railing design is considered inadequate even for the automobile portion of the vehicle spectrum.



NEW HAMPSHIRE
BRIDGE RAIL

Test No. 3451-12
 Date 12/14/78
 Rail 120.6 mm x 101.6 mm
 (4 3/4" x 4")
 semi-ellipse Alum. rail
 Post AASHTO BR2, Alum, Type D
 Post Spacing 2.44 m (8.00 ft)
 Post to Anchor Spacing 3.05 m (10.00 ft)
 Length of Installation 30.48 m (100.00 ft)
 Beam Rail Deflection
 Max. Dynamic 0.00 m (0.00 ft)
 Max. Permanent 0.00 m (0.00 ft)
 Vehicle Damage
 TAD 1RFQ5
 SAE 01RYAW4

Vehicle 1974 Honda
 Vehicle Weight 885 kg (1950 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 98.0 kph. (60.9 mph)
 Impact Angle 15.0 deg
 Exit Speed Vehicle Rolls
 Exit Angle 0 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -12.8 g
 Transverse 9.5 g
 Vertical -3.6 g

Figure 36. Summary of Results for Test 3451-12.

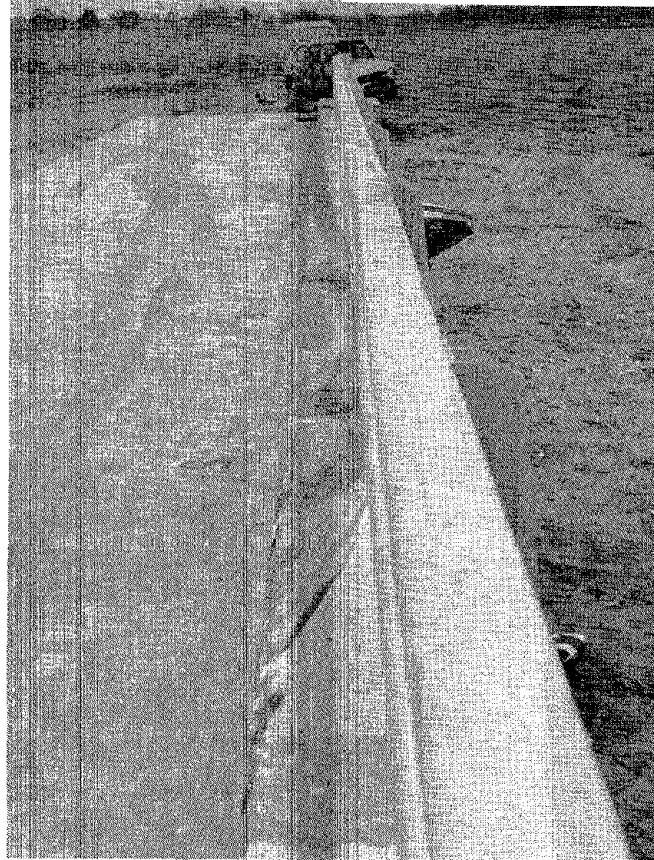
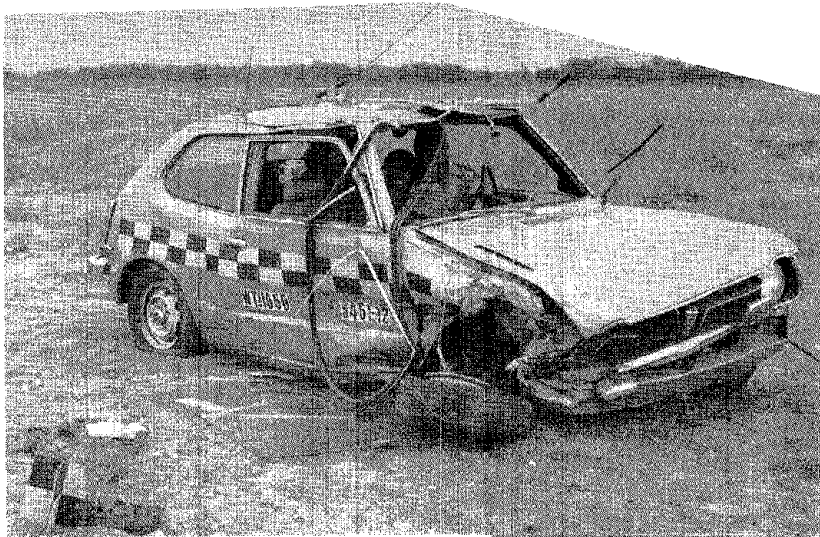
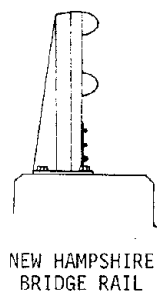
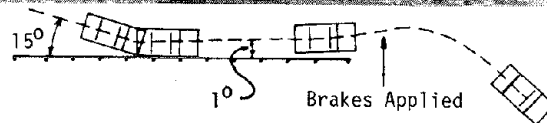
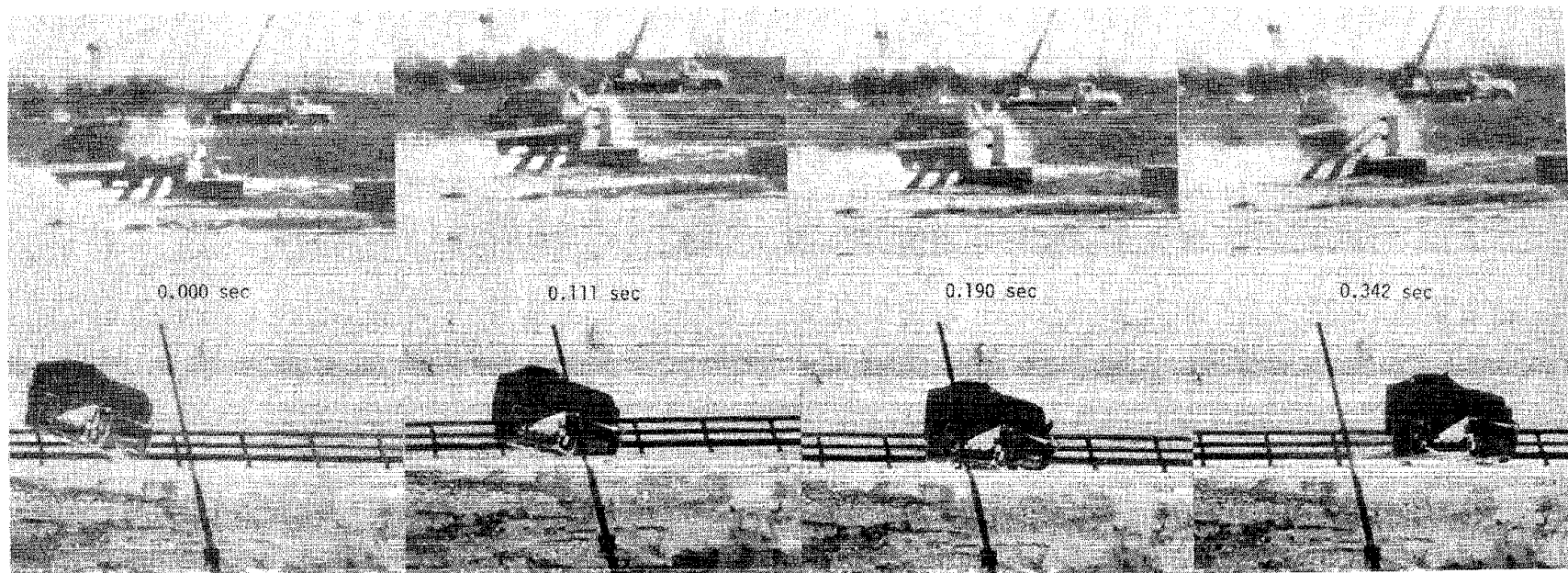


Figure 37. Photographs of Vehicle and Railing After Test 3451-12.



Test No. 3451-13
 Date 12/5/78
 Rail 120.6 mm x 101.6 mm
 (4 3/4" x 4")
 semi-ellipse Alum. rail
 Post AASHTO BR2, Alum, Type D
 Post Spacing 2.44 m (8.00 ft)
 Post to Anchor Spacing 3.05 m (10.00 ft)
 Length of Installation 30.48 m (100.00 ft)
 Beam Rail Deflection
 Max. Dynamic 0.00 m (0.00 ft)
 Max. Permanent 0.00 m (0.00 ft)
 Vehicle Damage
 TAD 1RFQ5
 SAE 01RFEW4

Vehicle 1974 Vega
 Vehicle Weight 1261 kg (2780 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 94.0 kph (58.4 mph)
 Impact Angle 15.0 deg
 Exit Speed 66.5 kph (41.3 mph)
 Exit Angle 1.0 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -7.1 g
 Transverse 9.0 g
 Vertical -2.5 g

Figure 38 . Summary of Results for Test 3451-13.

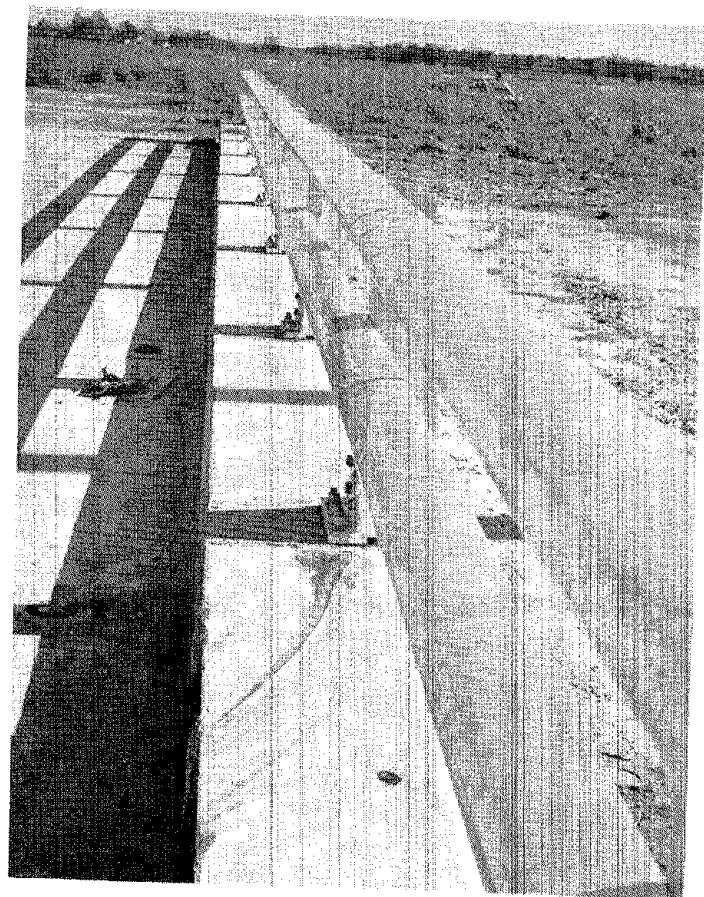


Figure 39. Photographs of Vehicle and Railing After Test 3451-13.

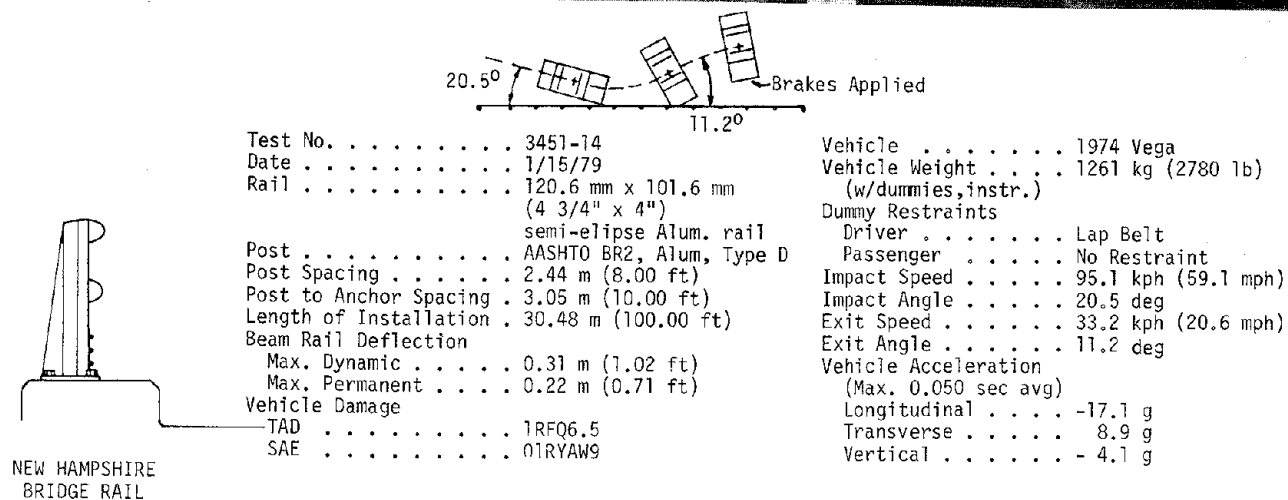
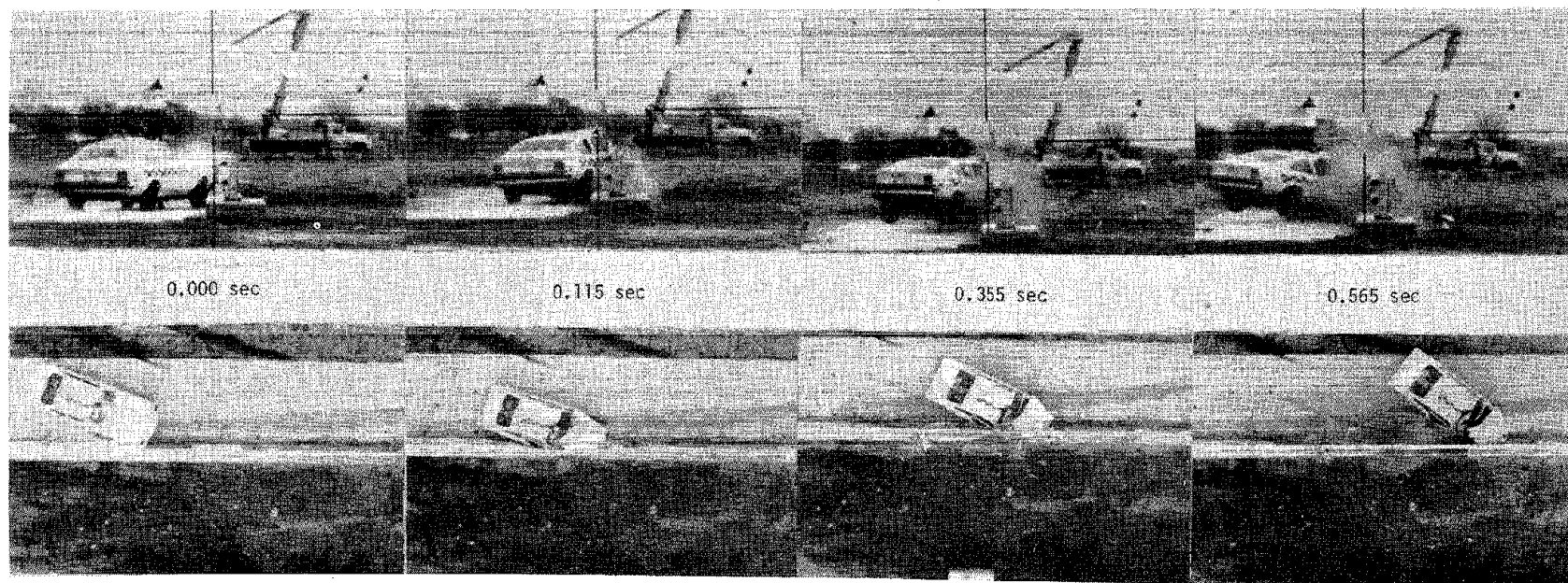


Figure 40. Summary of Results for Test 3451-14.

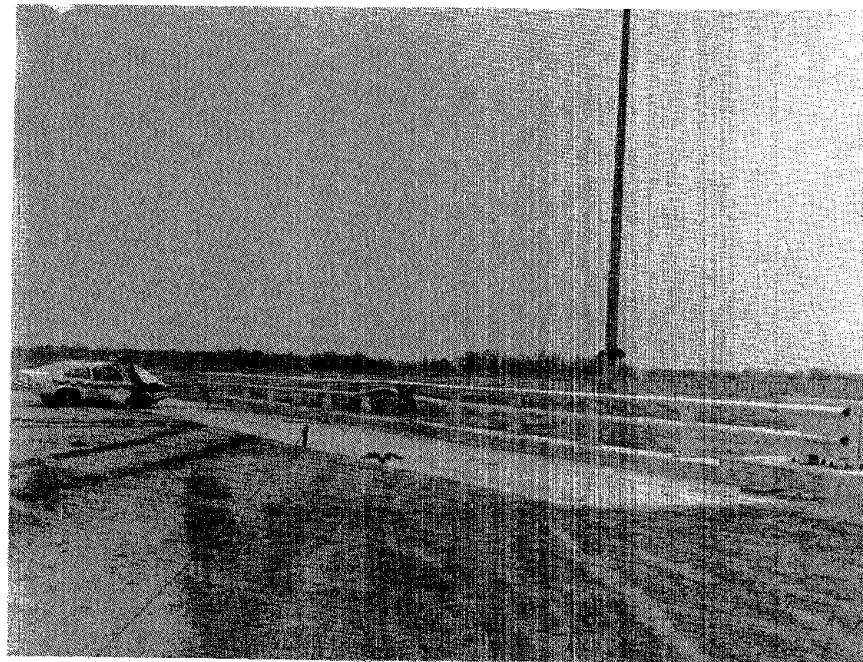
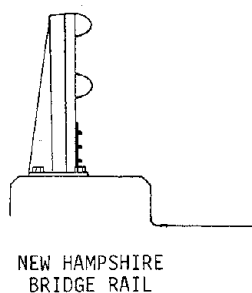
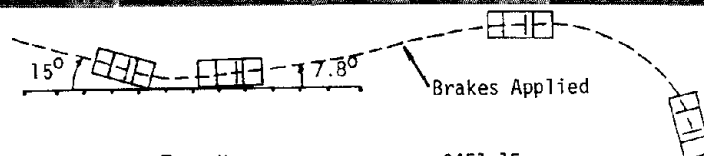
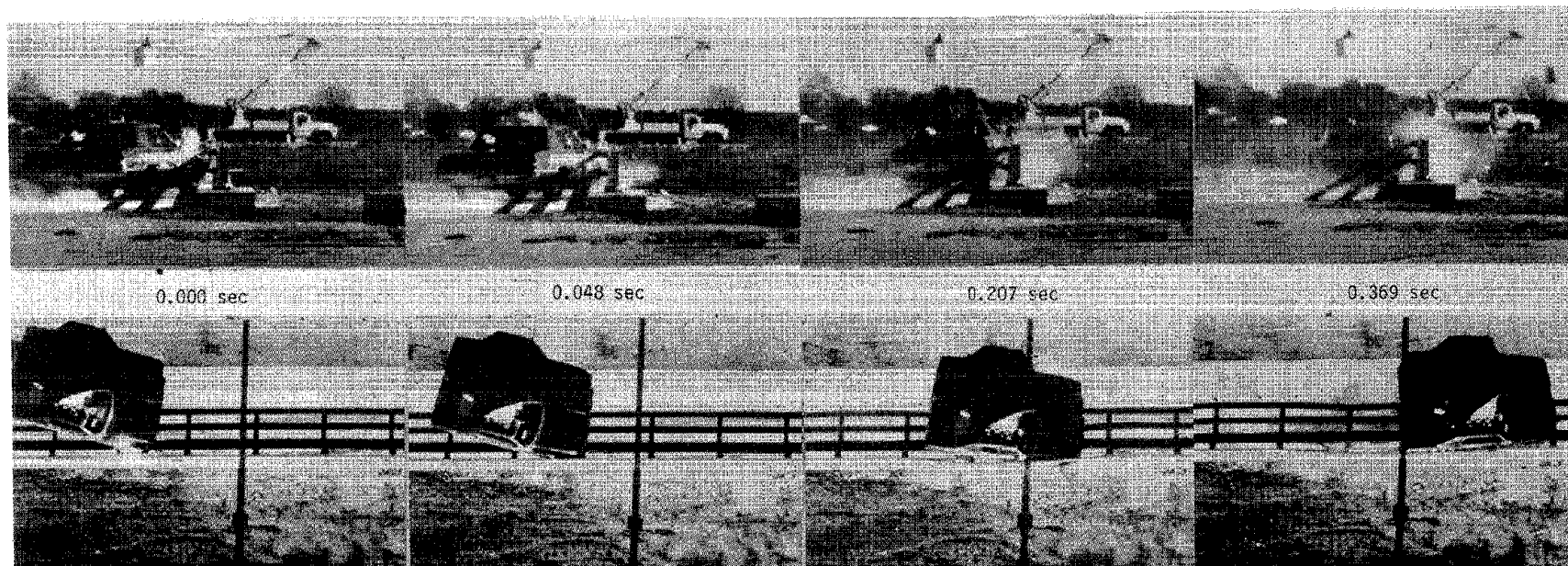


Figure 41 . Photographs of Vehicle and Railing After
Test 3451-14.



Test No. 3451-15
 Date 12/12/78
 Rail 120.6 mm x 101.6 mm
 (4 3/4" x 4")
 semi-ellipse Alum. rail
 Post AASHTO BR2, Alum, Type D
 Post Spacing 2.44 m (8.00 ft)
 Post to Anchor Spacing 3.05 m (10.00 ft)
 Length of Installation 30.48 m (100.00 ft)
 Beam Rail Deflection
 Max. Dynamic 0.27 m (0.89 ft)
 Max. Permanent 0.14 m (0.46 ft)
 Vehicle Damage
 TAD 1RFQ5
 SAE 01RFEE3

Vehicle 1974 Plymouth
 Vehicle Weight 2118 kg (4670 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 95.3 kph (59.2 mph)
 Impact Angle 15.0 deg
 Exit Speed 73.2 kph (45.5 mph)
 Exit Angle 7.8 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -5.1 g
 Transverse 6.0 g
 Vertical -3.2 g

Figure 42. Summary of Results for Test 3451-15.

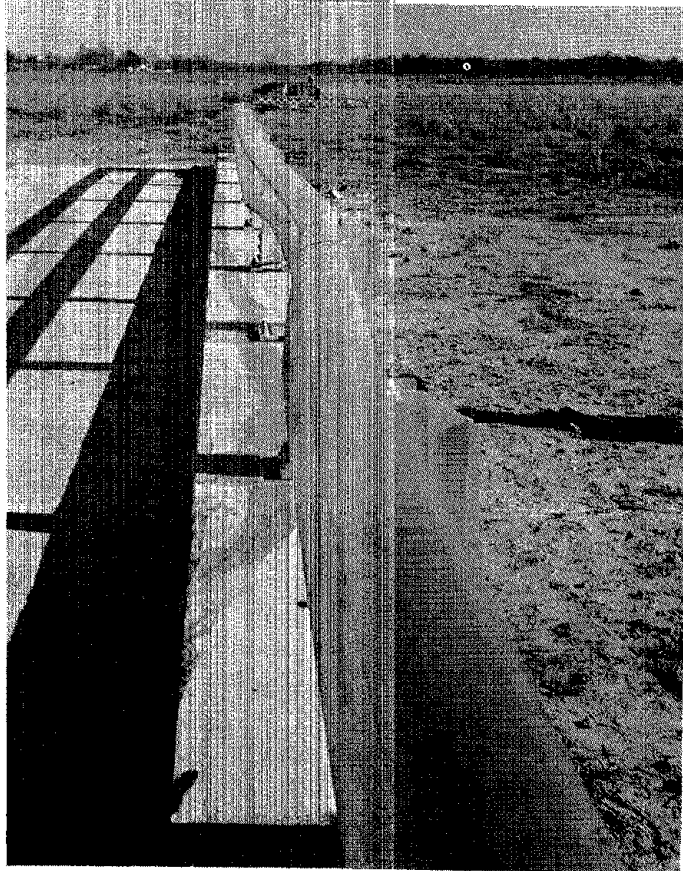


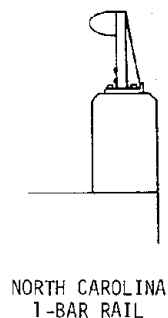
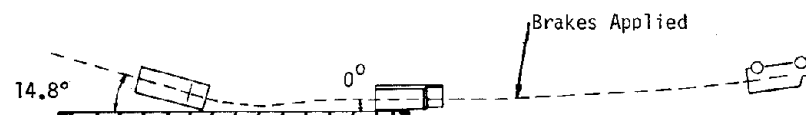
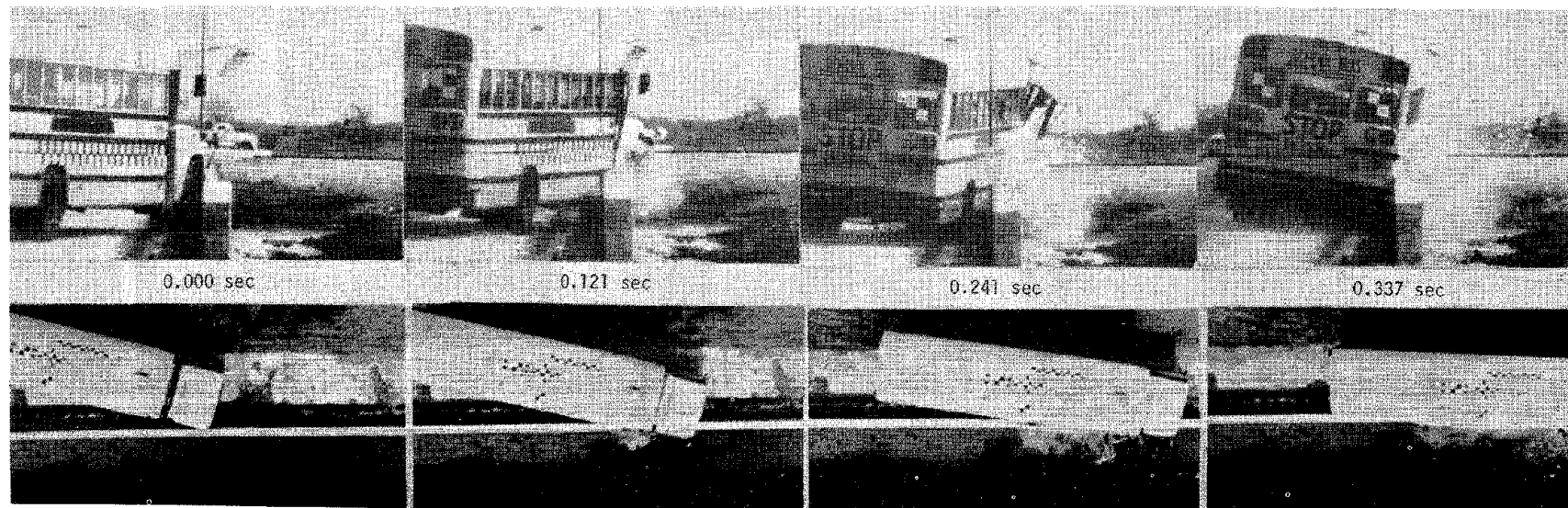
Figure 43. Photographs of Vehicle and Railing After Test 3451-15.

Results for North Carolina Railing: Only one test, a 9,036 kg (19,920 lb) school bus at 92.2 kph (57.3 mph) and 14.8 deg, was conducted on the North Carolina railing. The railing strength was adequate to contain and redirect the vehicle; however, the vehicle rolled onto the railing and slid off the end of the installation coming to rest on its side.

One of the features of this railing design that is considered undesirable from a crashworthiness point of view is the end treatment (Figure 45). This aspect of the design was not addressed in the testing reported herein; however, it is known from other testing that impacts on the concrete abutment by vehicles approaching the railing would result in unsatisfactory performance.

Some states have installed approach guardrail with bridge railings of similar design and have provided apparently adequate transitions between the two systems. In some installations, the approach railing is simply terminated at the abutment with no connection or transition and this is deemed inadequate.

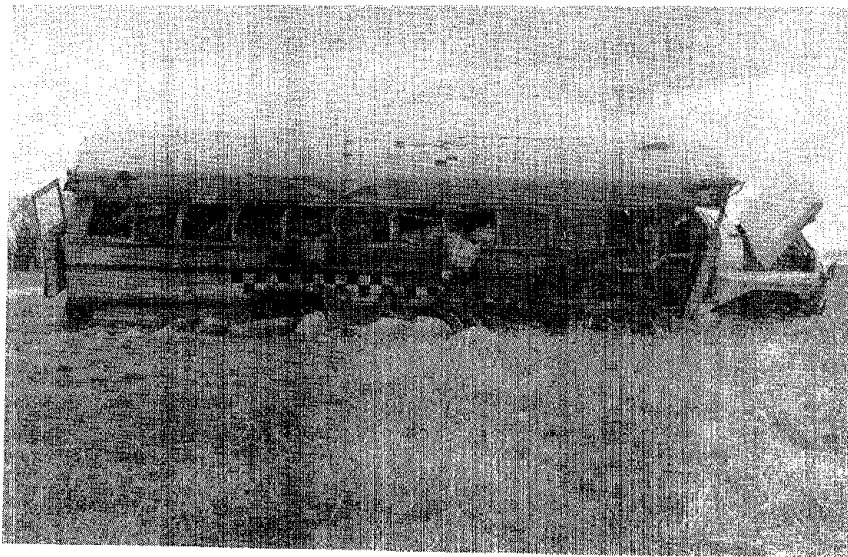
The concrete abutment may also snag vehicles impacting the bridge railing near the abutment while exiting the structure. In some installations the metal rail element is terminated, with no connection, at the abutment and may deflect laterally allowing a vehicle to snag on the abutment. This behavior was not strongly in evidence in the school bus test because the impact point was sufficiently removed from the abutment that the vehicle was well redirected when it approached the abutment.



Test No. 3451-23
 Date 2/21/79
 Rail 120.6 mm x 101.6 mm
 (4 3/4" x 4")
 semi-ellipse Alum. rail
 Post AASHTO BRL, Alum, Type C
 Post Spacing 2.0 m (6.5 ft)
 Post to Anchor Spacing 0.4 m (1.2 ft)
 Length of Installation 30.5 m (100 ft)
 Beam Rail Deflection
 Max. Dynamic 0.4 m (1.5 ft)
 Max. Permanent 0.3 m (0.9 ft)
 Vehicle Damage
 TAD 1RFQ6
 SAE OIRYEE2
 Maximum Roll Vehicle rolled onto side

Vehicle 1969 Ford School Bus
 Vehicle Weight 9036 kg (19,920 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passengers No Restraint
 Impact Speed 92.2 kph (57.3 mph)
 Impact Angle 14.8 deg
 Exit Speed N.A. Rolls & Exits View
 Exit Angle 0 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -1.9 g
 Transverse 4.9 g
 Vertical -1.8 g

Figure 44 . Summary of Results for Test 3451-23.



Bus after being
uprighted

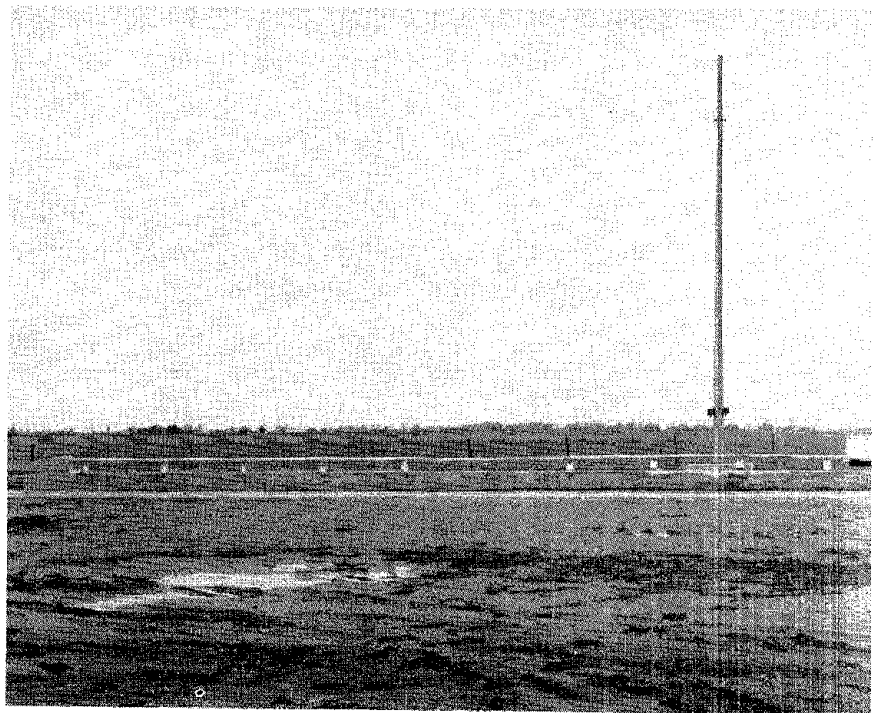


Figure 45. Photographs of Vehicle and Railing After
Test 3451-23.

Results for Indiana Type 5A Railing: A series of four tests (Numbers 24 through 27) was conducted on the Indiana Type 5A railing, and Test 28 was conducted on the Modified 5A railing. In Test 24, with a Honda Civic, the actual impact angle was only 12.5 deg and a very smooth redirection was achieved. In the other tests, the vehicle front wheel underrode the lower rail element and snagged on one or more posts. The vehicle bumper also had a tendency to override the lower rail element. This tendency was aggravated to some extent in Test 28.

Geometrics of both the type 5A and the modified 5A were within AASHTO Specification (17) limits but its performance in 15 deg and 20 deg tests with automobiles was unsatisfactory.

In the 25 deg test with a 2,043 kg (4,500 lb) automobile (Test 26), the railing failed to contain and redirect the vehicle. The inadequate geometrics of the railing allowed the vehicle to penetrate deeply enough into the railing to snag on a post. Structural failure of the post baseplate then occurred and further penetration of the railing followed.

It was at this point that the weak failure mode in the baseplate was demonstrated. The reduced thickness in the middle portion of the baseplate allowed the toe of the baseplate to bend downward. This, in turn, generated excessively high combined tensile and bending stresses at the juncture of the baseplate and sail with structural fracture occurring at this juncture rather than in the rivets.

After this test series was completed, the Aluminum Association decided to further investigate the performance of this railing design and to explore means of improving its performance. Successive modifications were made and subjected to tests (15).

The first modification was to make the baseplate a constant 22 mm (7/8 in.) thick. In a strength test on this design, the vehicle penetrated the railing in a fashion similar to that observed earlier (Figure 56). The failure mode in the posts was stripping of the aluminum anchor nuts.

These anchor nuts were replaced with steel nuts and the vehicle again penetrated the railing in a strength test (Figure 57). However, in this test the failure mode of the posts was shear of the rivets connecting the baseplate assembly to the post (Figure 58).

Results of these and the previous test showed a definite problem with geometrics of this railing design. To alleviate this problem, a new rail element with more frontal area was developed (Figure 59). This rail element also reduced the clear opening immediately above the deck to 0.33 m (13 in.). In a strength test with a 2,043 kg (4,500 lb) automobile, the railing performed satisfactorily (Figure 60). This railing was also tested with a Honda Civic at 97 kph (60 mph) and 20 deg (16). In this test, the vehicle front wheel did not snag and performance was considered satisfactory.

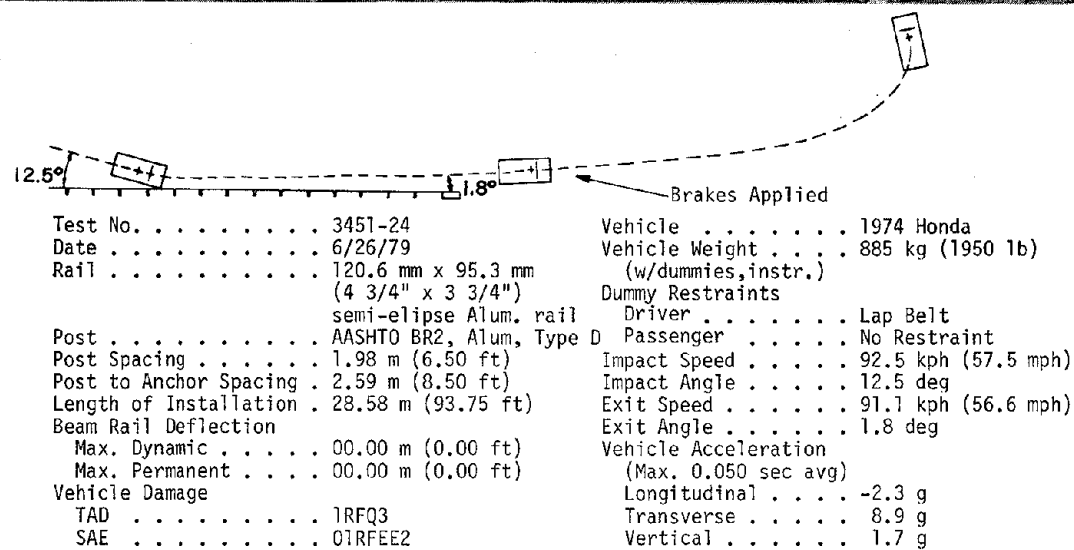
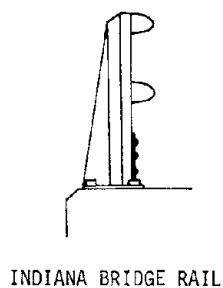


Figure 46. Summary of Results for Test 3451-24.

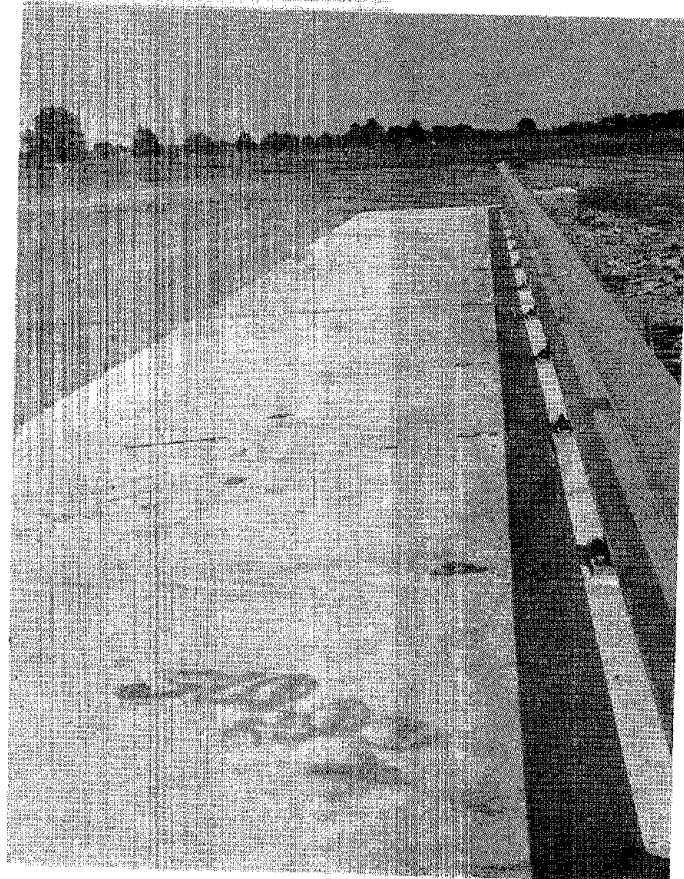
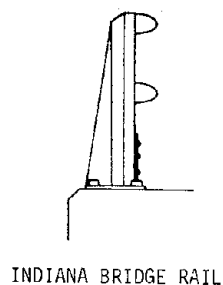
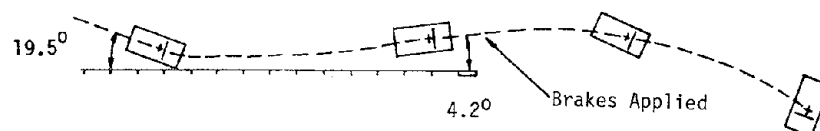
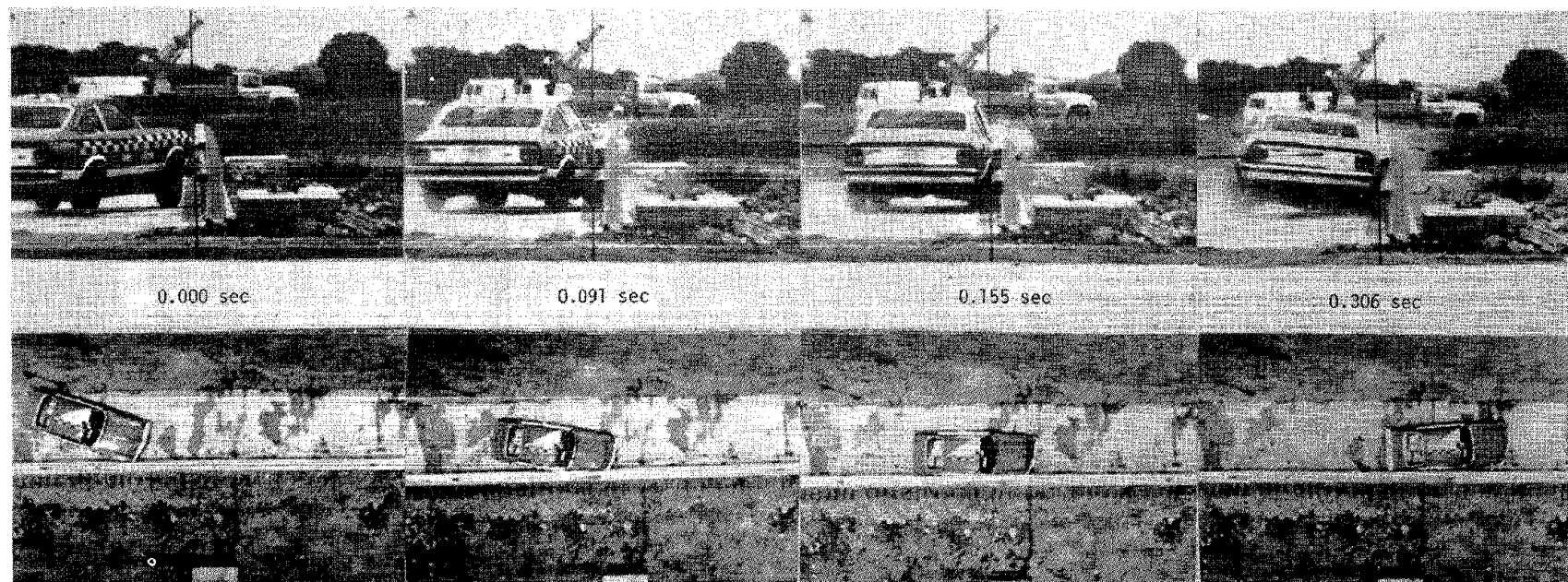


Figure 47. Photographs of Vehicle and Railing After Test 3451-24.



Test No. 3451-25
 Date 6/28/79
 Rail 120.6 mm x 95.3 mm
 (4 3/4" x 3 3/4")
 semi-ellipse Alum. rail
 Post AASHTO BR2, Alum, Type D
 Post Spacing 1.98 m (6.50 ft)
 Post to Anchor Spacing 2.59 m (8.50 ft)
 Length of Installation 28.58 m (93.75 ft)
 Beam Rail Deflection
 Max. Dynamic 0.08 m (0.26 ft)
 Max. Permanent 0.02 m (0.06 ft)
 Vehicle Damage
 TAD 1RFQ4.4
 SAE 01RFAE3

Vehicle 1974 Chevrolet Vega
 Vehicle Weight 1261 kg (2780 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 86.3 kph (53.6 mph)
 Impact Angle 19.5 deg
 Exit Speed 68.9 kph (42.8 mph)
 Exit Angle 4.2 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -4.9 g
 Transverse 6.6 g
 Vertical -3.2 g

Figure 48. Summary of Results for Test 3451-25.

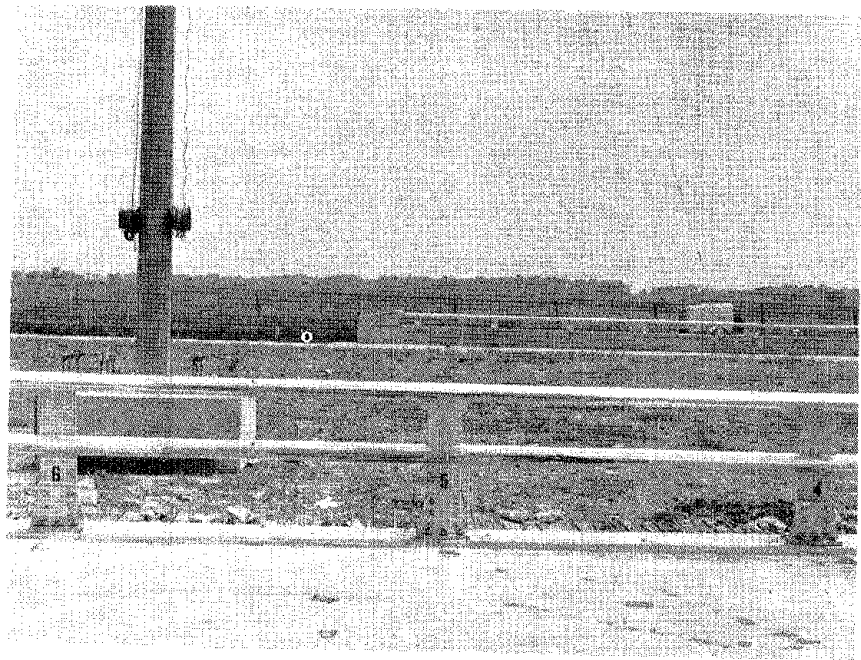
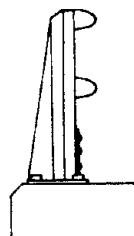
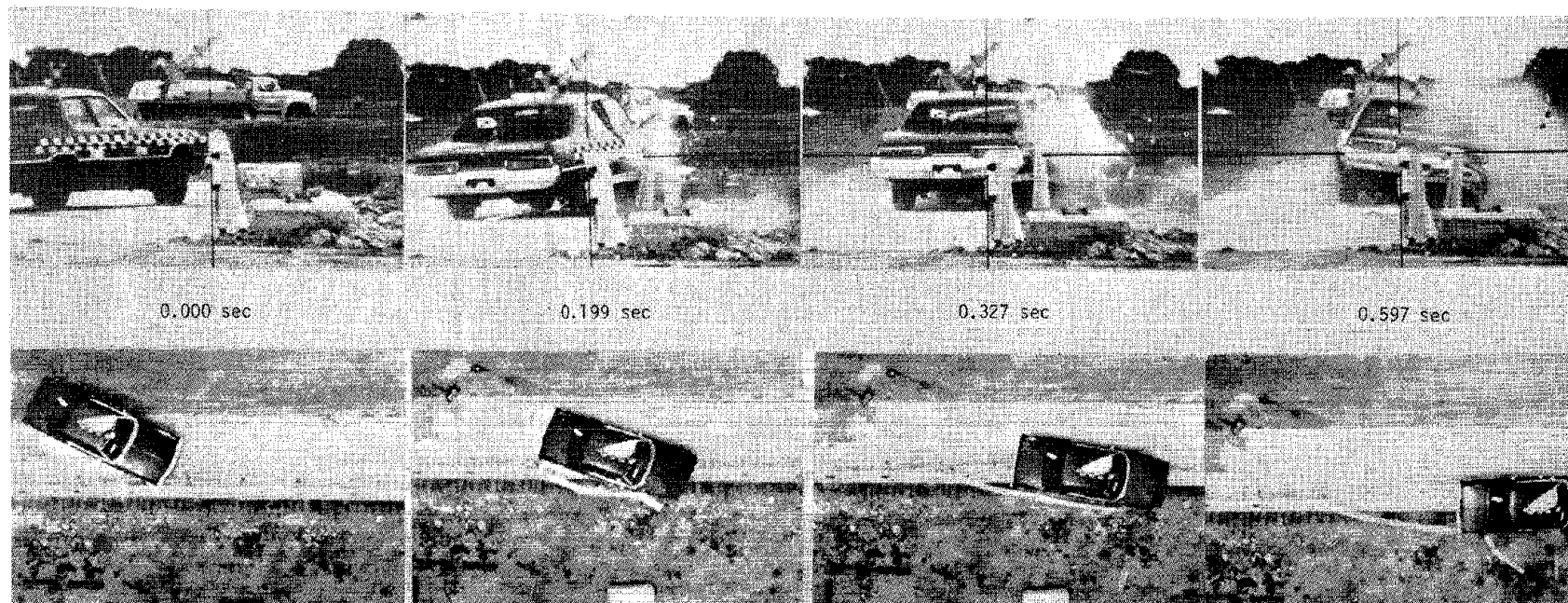


Figure 49. Photographs of Vehicle and Railing After Test 3451-25.



INDIANA BRIDGE RAIL

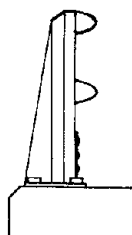
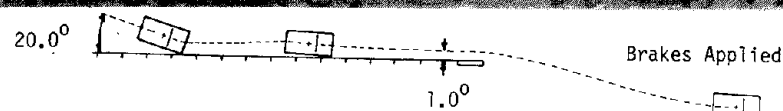
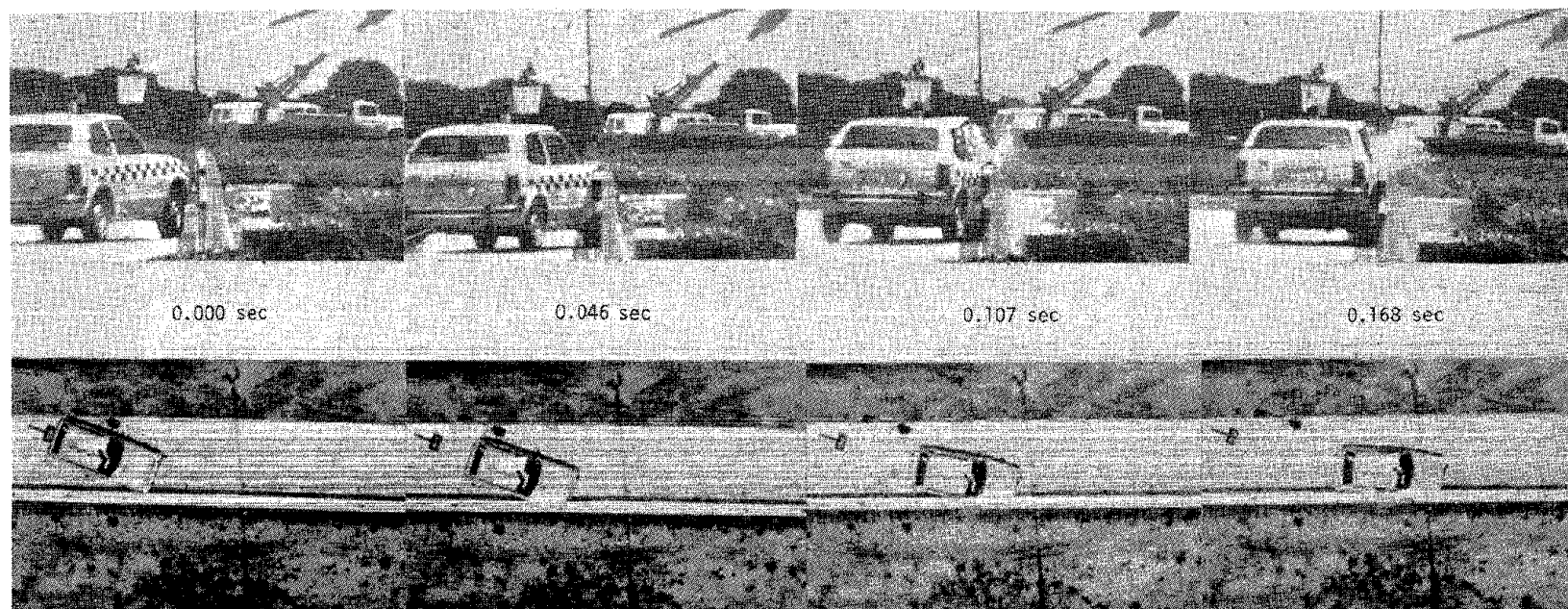
Test No. 3451-26
 Date 7/3/79
 Rail 120.6 mm x 95.3 mm
 (4 3/4" x 3 3/4")
 semi-ellipse Alum. rail
 Post AASHTO BR2, Alum, Type D
 Post Spacing 1.98 m (6.50 ft)
 Post to Anchor Spacing 2.59 m (8.50 ft)
 Length of Installation 28.58 m (93.75 ft)
 Beam Rail Deflection
 Max. Dynamic 1.13 m (3.72 ft)
 Max. Permanent Rail Broke
 Vehicle Damage
 TAD IFR6
 SAE 01FYEW9

Vehicle 1974 Plymouth
 Vehicle Weight 2118 kg (4670 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 99.1 kph (61.6 mph)
 Impact Angle 25.8 deg
 Exit Speed 0.0 kph (0.0 mph)
 Rebound Angle 4.0 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -14.8 g
 Transverse 6.5 g
 Vertical 4.3 g

Figure 50 . Summary of Results for Test 3451-26.



Figure 51. Photograph of Vehicle After Test 3451-26.



INDIANA BRIDGE RAIL

Test No.	3451-27	Vehicle	1974 Honda
Date	7/31/79	Vehicle Weight	975 kg (2150 lbs)
Rail	120.6 mm x 95.3 mm	(w/dummies, instr.)	
	(4 3/4" x 3 3/4")	Dummy Restraints	
	semi-ellipse Alum. Rail	Driver	Lap Belt
Post	AASHTO BR2, Alum, Type D	Passenger	No Restraint
Post Spacing	1.98 m (6.50 ft)	Impact Speed	88.2 kph (54.8 mph)
Post to Anchor Spacing . .	2.59 m (8.50 ft)	Impact Angle	20.0 deg
Length of Installation .	28.58 m (93.75 ft)	Exit Speed	66.9 kph (41.6 mph)
Beam Rail Deflection		Exit Angle	1.0 deg
Max. Dynamic	0.07 m (0.24 ft)	Vehicle Acceleration	
Max. Permanent	0.02 m (0.05 ft)	(Max. 0.050 sec avg)	
Vehicle Damage		Longitudinal	-9.2 g
TAD	1RF06	Transverse	10.3 g
SAE	01RYEE3	Vertical	2.3 g

Figure 52. Summary of Results for Test 3451-27.

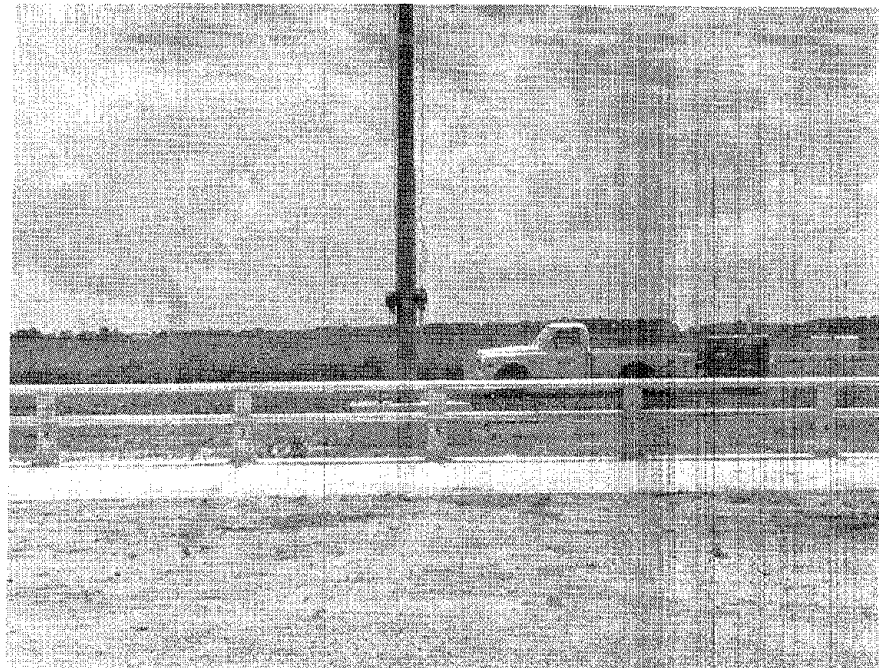
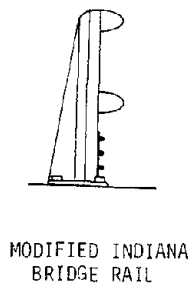
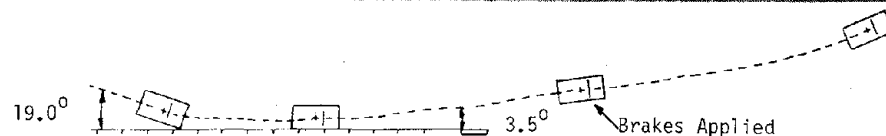
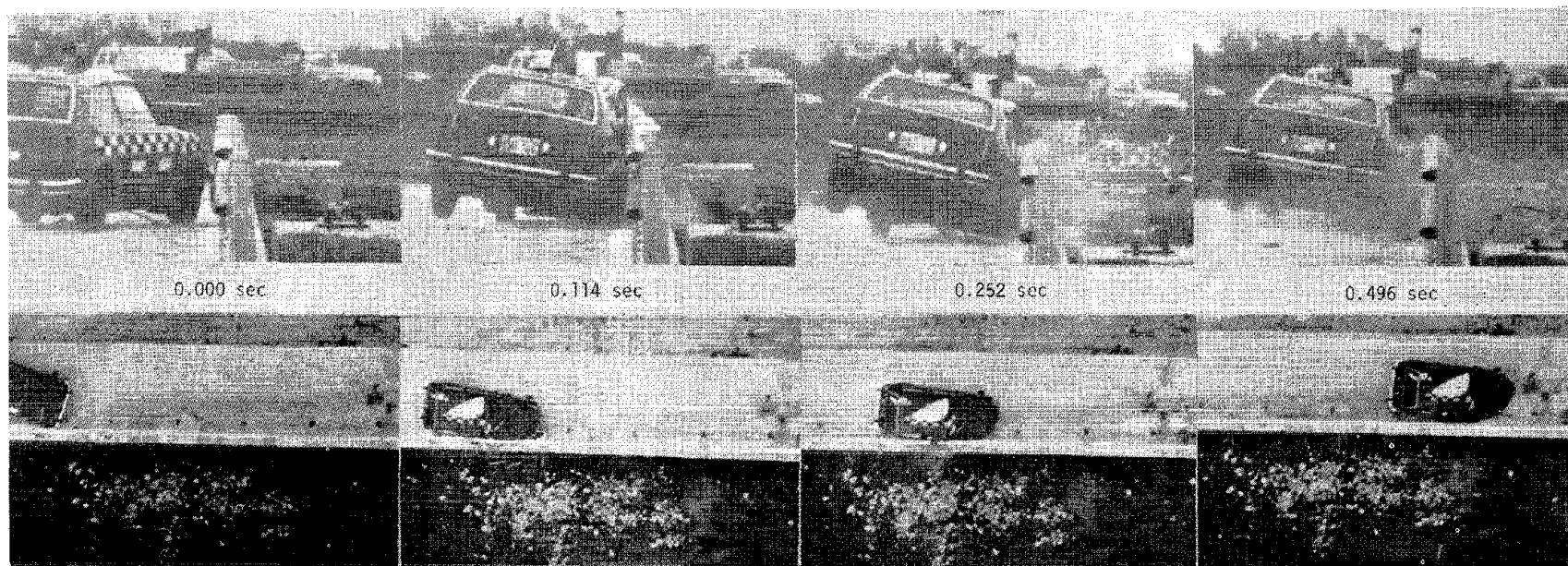


Figure 53. Photographs of Vehicle and Railing After Test 3451-27.



Test No. 3451-28
 Date 2/11/80
 Rail 120.6 mm x 95.3 mm
 (4 3/4" x 3 3/4")
 semi-ellipse Alum. Rail
 Post AASHTO BR2, Alum, Type D
 Post Spacing 1.98 m (6.50 ft)
 Post to Anchor Spacing 2.59 m (8.50 ft)
 Length of Installation 28.58 m (93.75 ft)
 Beam Rail Deflection
 Max. Dynamic 0.04 m (0.13 ft)
 Max. Permanent 0.01 m (0.04 ft)
 Vehicle Damage
 TAD 1RF07
 SAE 01RYAW5

Vehicle 1974 Honda
 Vehicle Weight 930 kg (2050 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 89.2 kph (55.4 mph)
 Impact Angle 19.0 deg
 Exit Speed 57.1 kph (35.5 mph)
 Exit Angle 3.5 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -13.6 g
 Transverse 10.2 g
 Vertical -4.1 g

Figure 54. Summary of Results for Test 3451-28.

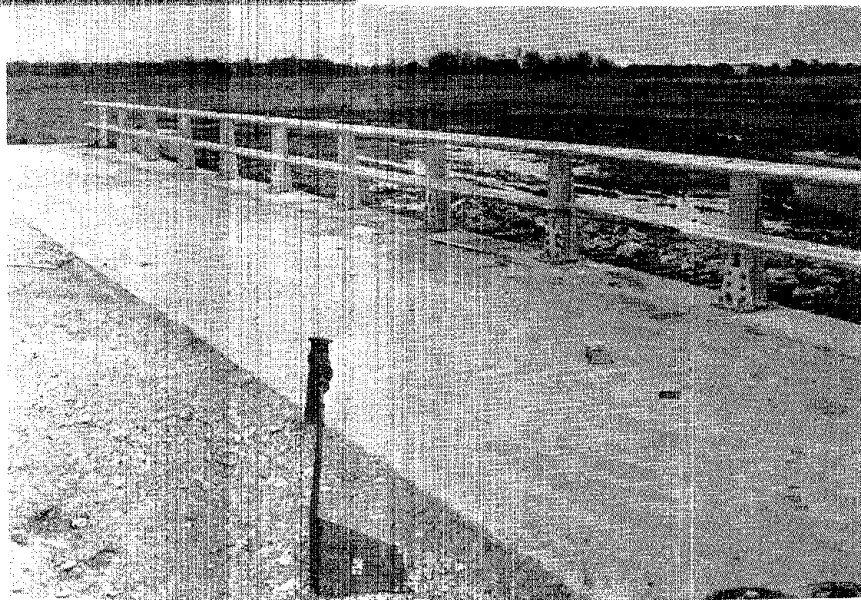


Figure 55. Photographs of Vehicle and Railing After Test 3451-28.

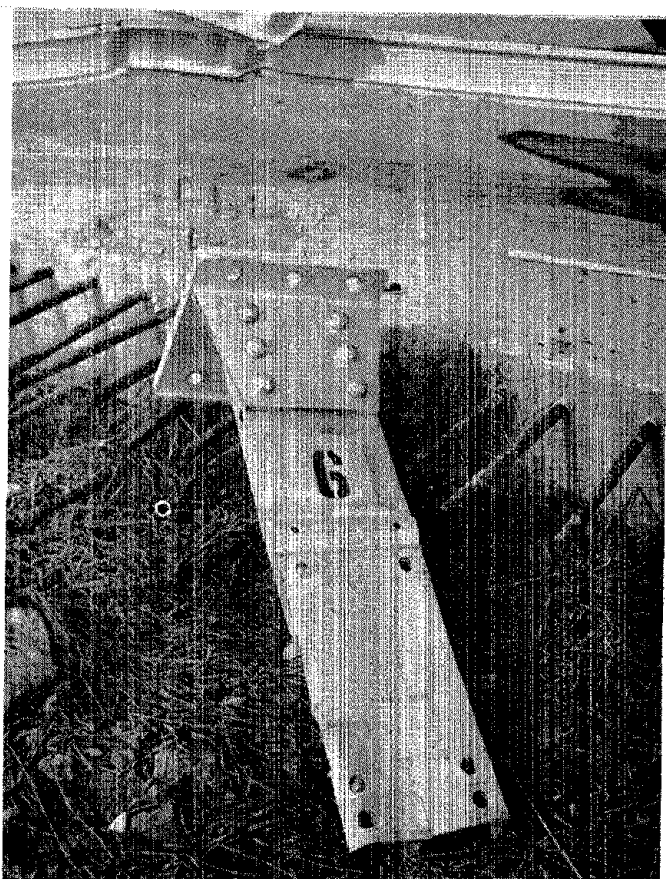
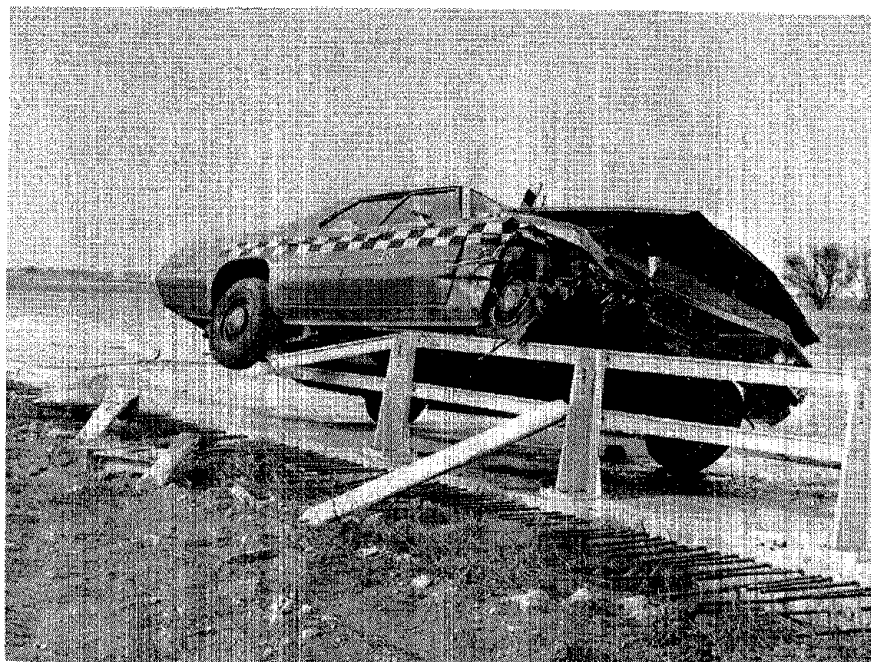


Figure 56. Vehicle and Post 9 After Test 4182-1 (Ref. 15).

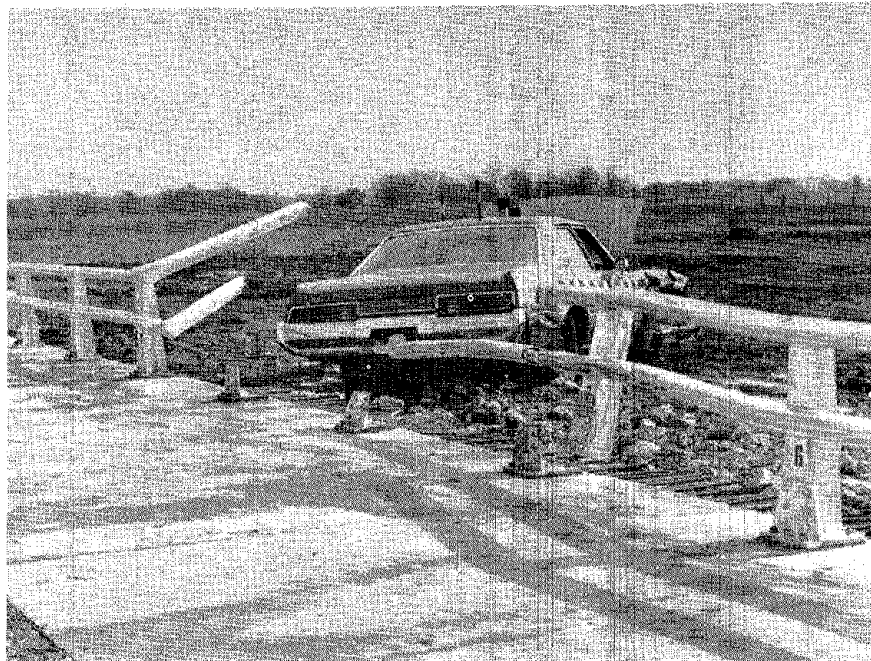
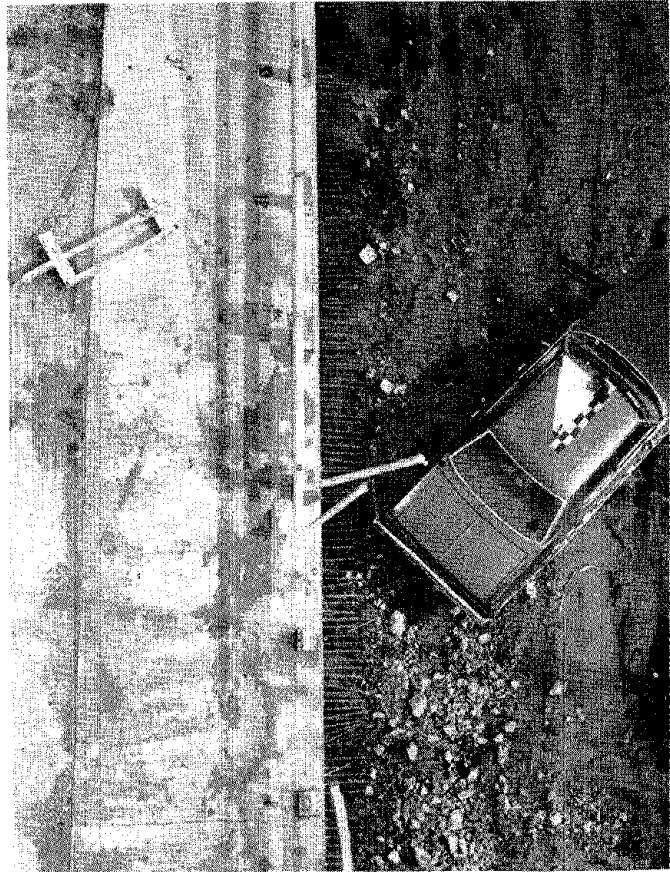


Figure 57. Overhead and Oblique
Views of Test Area
After Test 4182-2 (Ref. 15).

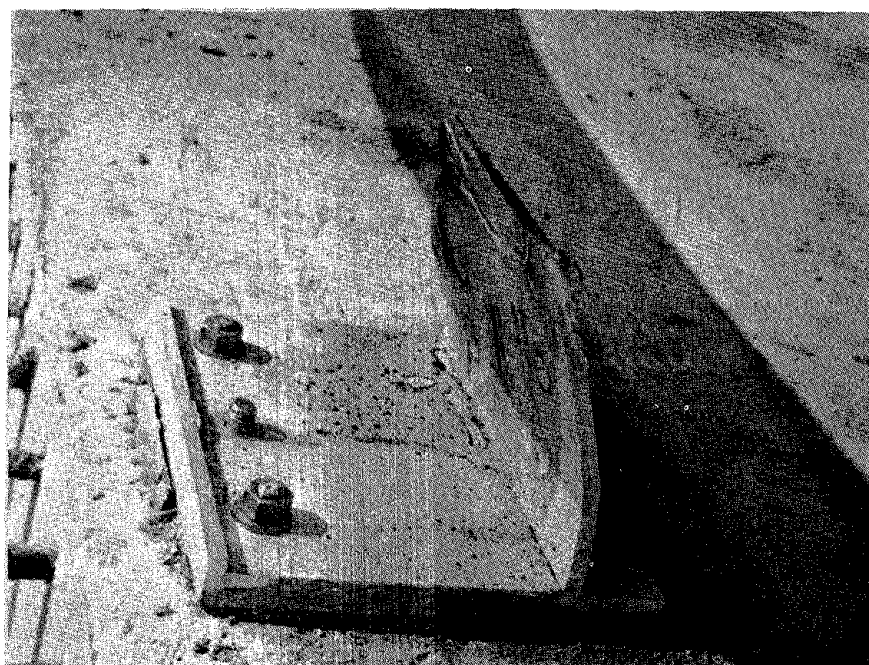
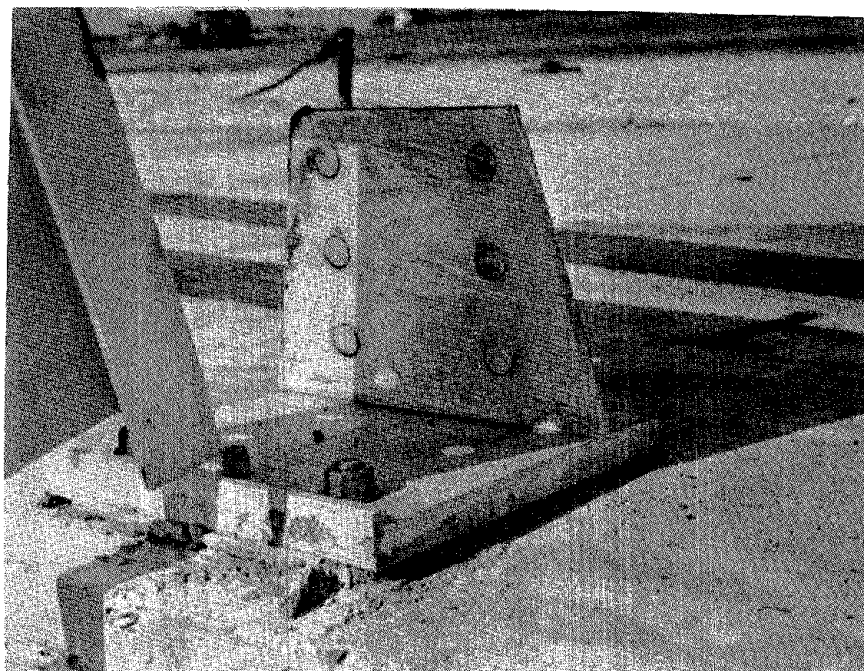
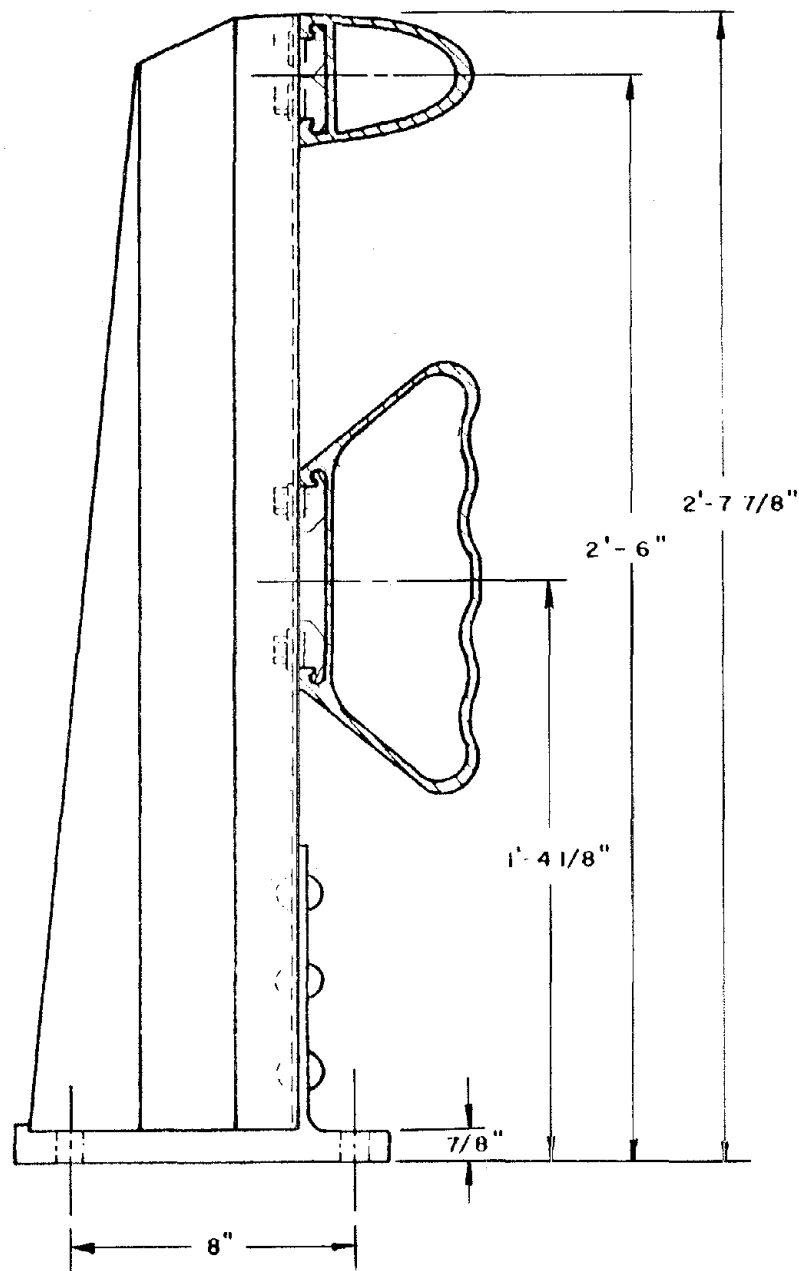


Figure 58. Sheared Rivets on Post Mountings
After Test 4182-2 (Ref. 15).



1 in. = 25.40 mm
 1 ft = 0.305 m

Figure 59. The Modified Indiana Type 5A Rail with the Magnode Tru-Beam.

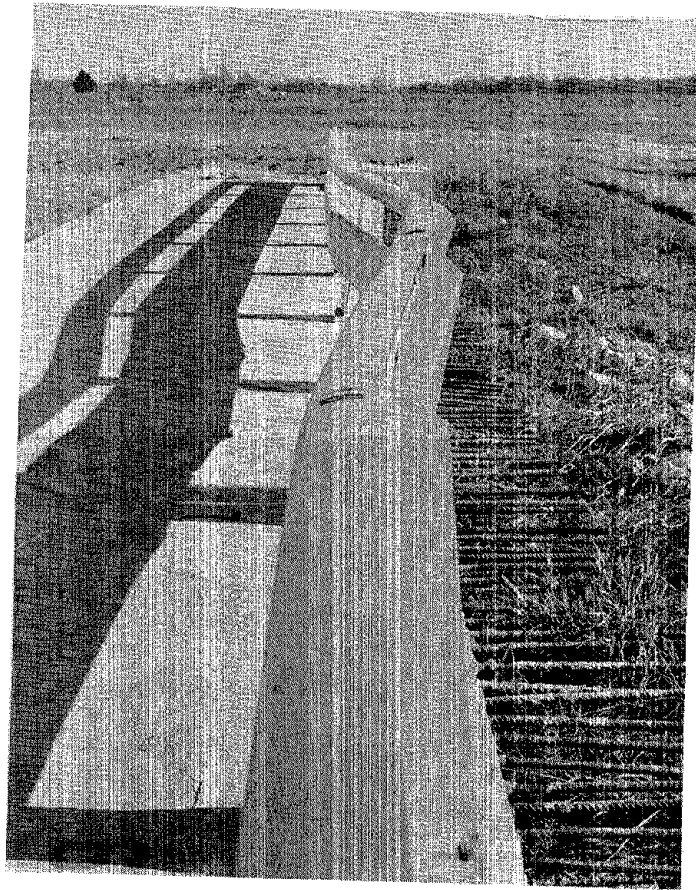
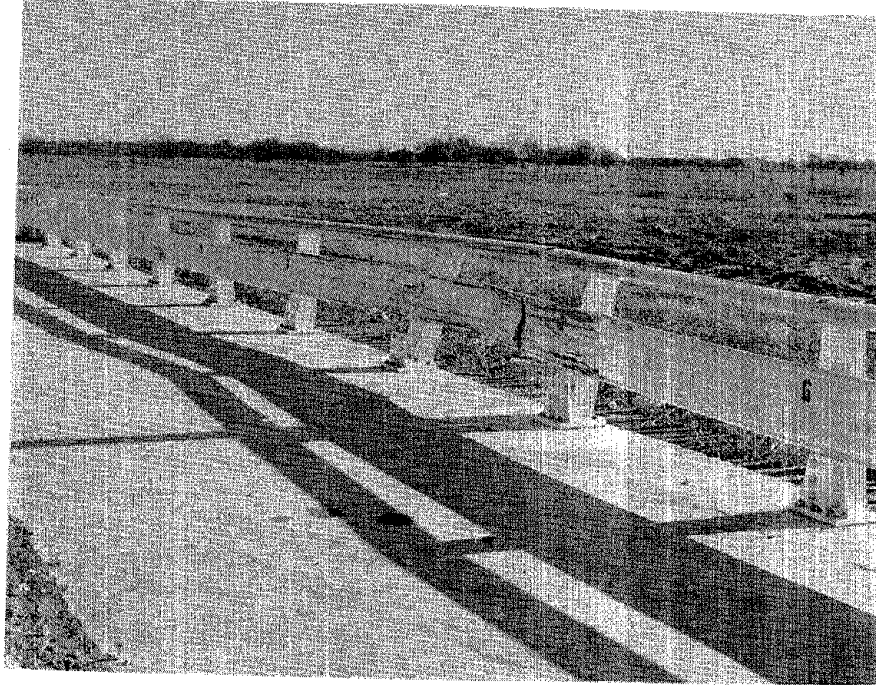


Figure 60. Damage to Railing After Test 4182-3 (Ref. 15).

Results for Instrumented Wall: A series of nine tests (Numbered 29 through 37) was conducted on the instrumented wall. The purpose of this series of tests was to determine baseline performance response of vehicles and to measure lateral forces imposed by a vehicle during a collision. The wall was instrumented with load cells and accelerometers to measure magnitude and location of forces perpendicular to the face of the wall. For details of the design and calibration of instrumentation, the reader is referred to the "Testing Program" section of Appendix C.

Total height of the wall was 1.07 m (42 in.). This height was selected to provide acceptable roll stability for the larger vehicles, thereby providing for suitable measurement of collision forces in an acceptable redirection not complicated by vehicle stability problems.

In each test, force versus time data were obtained from each transducer. These data were combined to obtain traces of total lateral force imposed on the wall and location of the resultant of the total lateral force. Data from each of the individual, 3 m (10 ft) long segments in the wall were also analyzed in a similar fashion to aid in determining the distribution of force along the length of the wall.

Traces of total lateral force versus time shows that two impacts (or impulses) occurred in each test. This phenomena was more pronounced in tests with larger vehicles. Highest 0.050 sec average values of total lateral force were computed for each of the impacts--initial and final. Locations of the resultant forces at corresponding time intervals were then computed. Results of these analyses are summarized in Table 6 and the data are further analyzed in Appendices D and E.

Table 6. Vertical Location and Magnitude of Forces from Instrumented Wall.

Test Number and Conditions	Initial Impact Force		Final Impact Force	
	Magnitude (kips)	Height (in.)	Magnitude (kips)	Height (in.)
3451-29 1,970 lb/59.0 mph/15.5 deg	18.4	16.0	8.4	18.7
3451-37 2,090 lb/58.5 mph/21.0 deg	21.1	18.8	13.1	20.7
3451-30 2,800 lb/58.3 mph 14.8 deg	18.5	17.7	13.9	16.0
3451-31 2,830 lb/56.0 mph 20.0 deg	22.0	20.0	22.5	21.0
3451-32 4,680 lb/54.6 mph/16.5 deg	52.5	21.2	28.3	24.1
3451-36 4,740 lb/59.8 mph/24.0 deg	59.9	21.9	28.3	22.5
3451-34 20,030 lb/57.6 mph/16.5 deg	63.7	28.5	73.8	33.0
3451-35 32,020 lb/56.9 mph/15.8 deg	85.0	26.3	211.0	28.0

1 lb = 0.454 kg
1 mph = 0.609 kph

1 kip = 4.448 kN
1 in. = 0.025 m

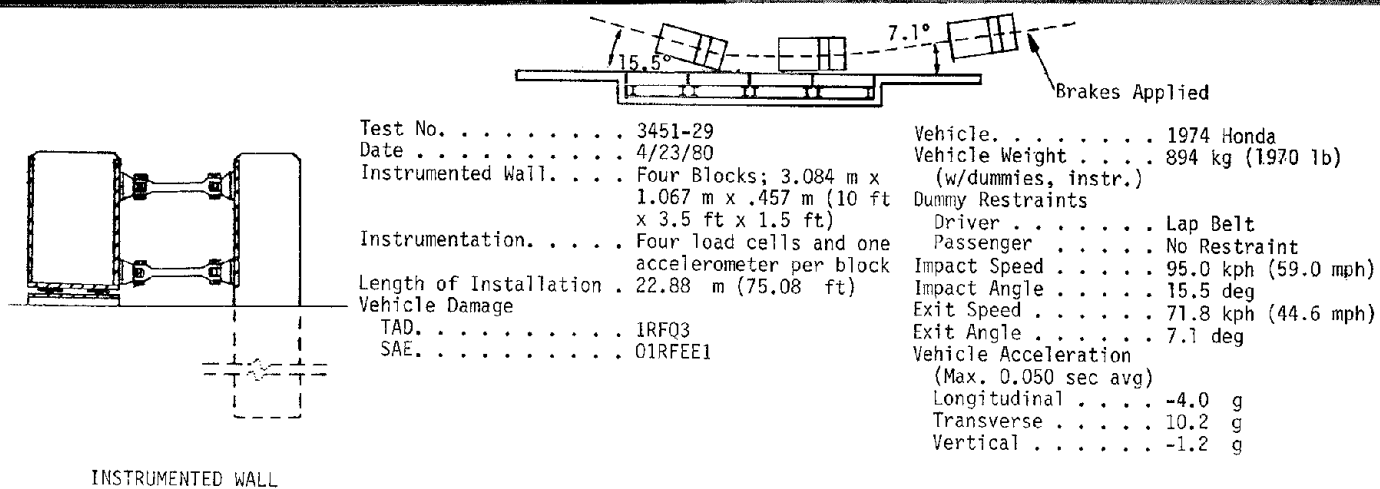
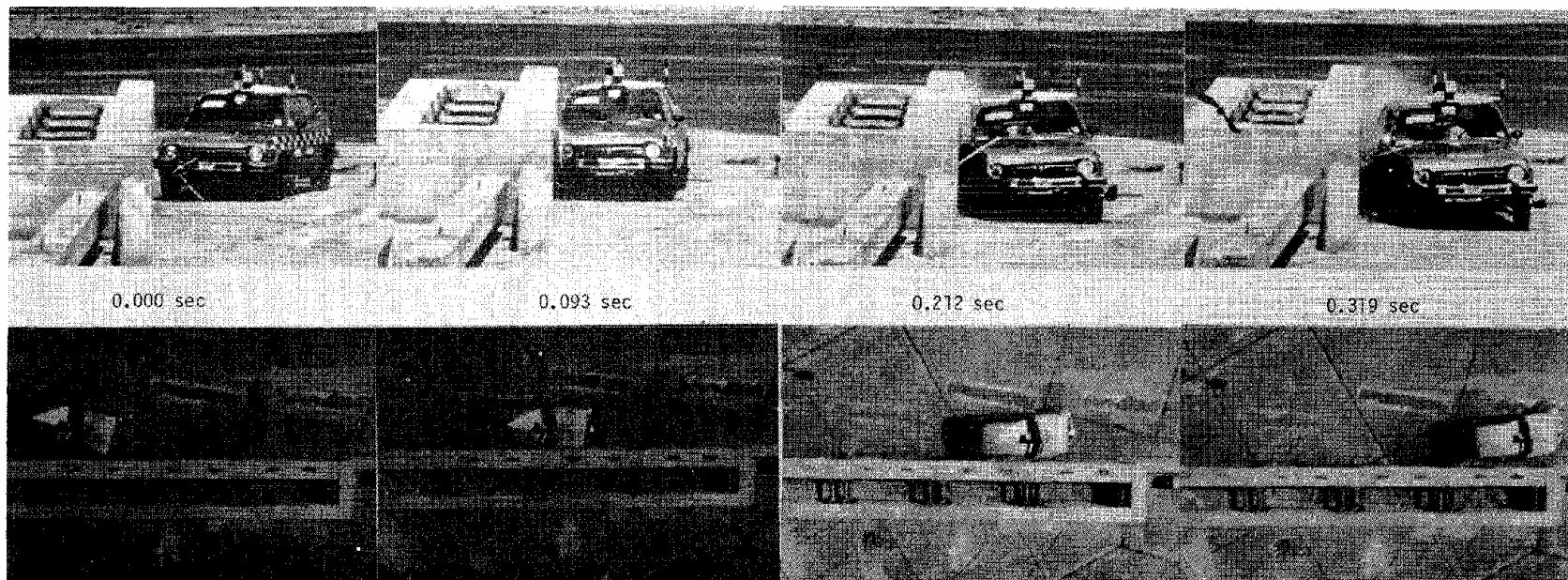


Figure 61. Summary of Results for Test 3451-29.

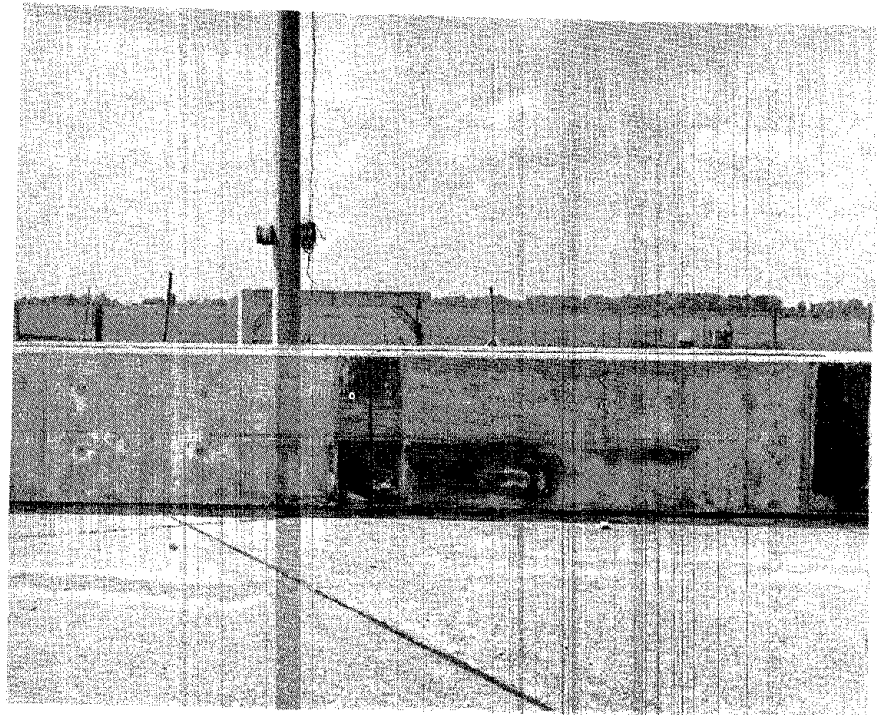
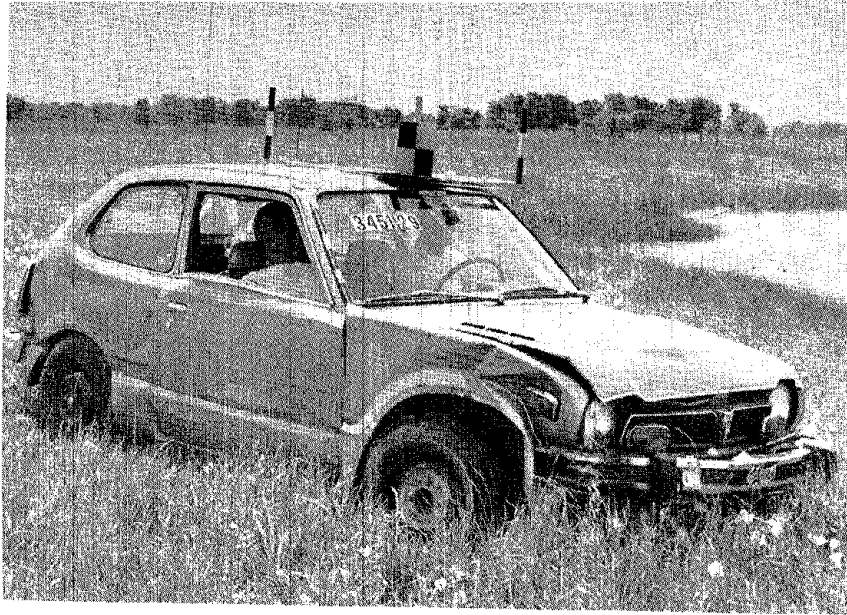


Figure 62. Photographs of Vehicle and Railing After Test 3451-29.

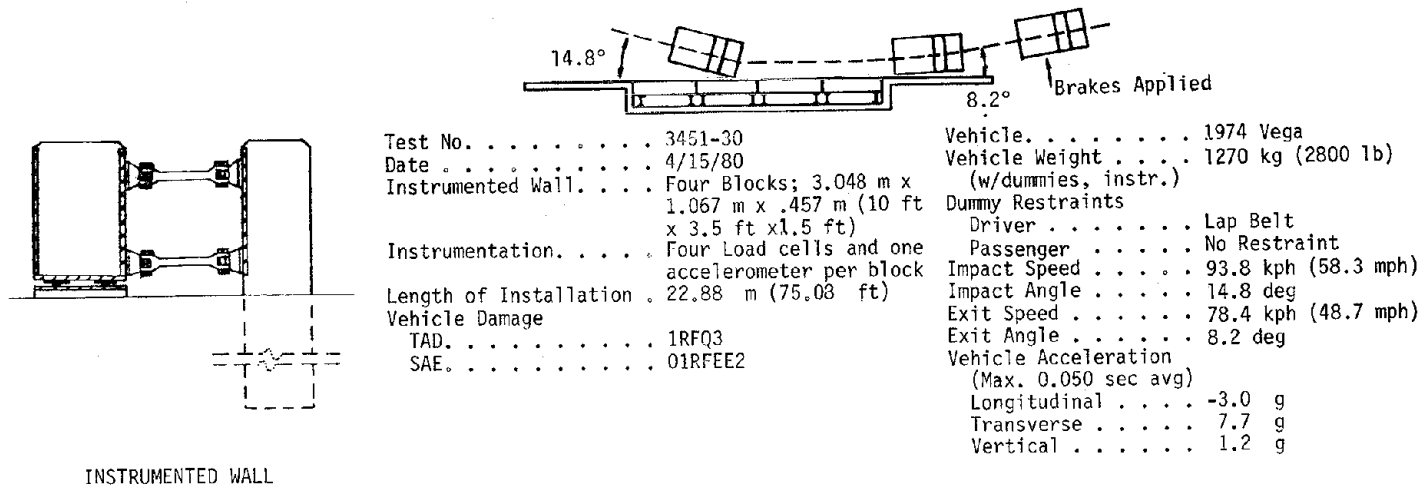
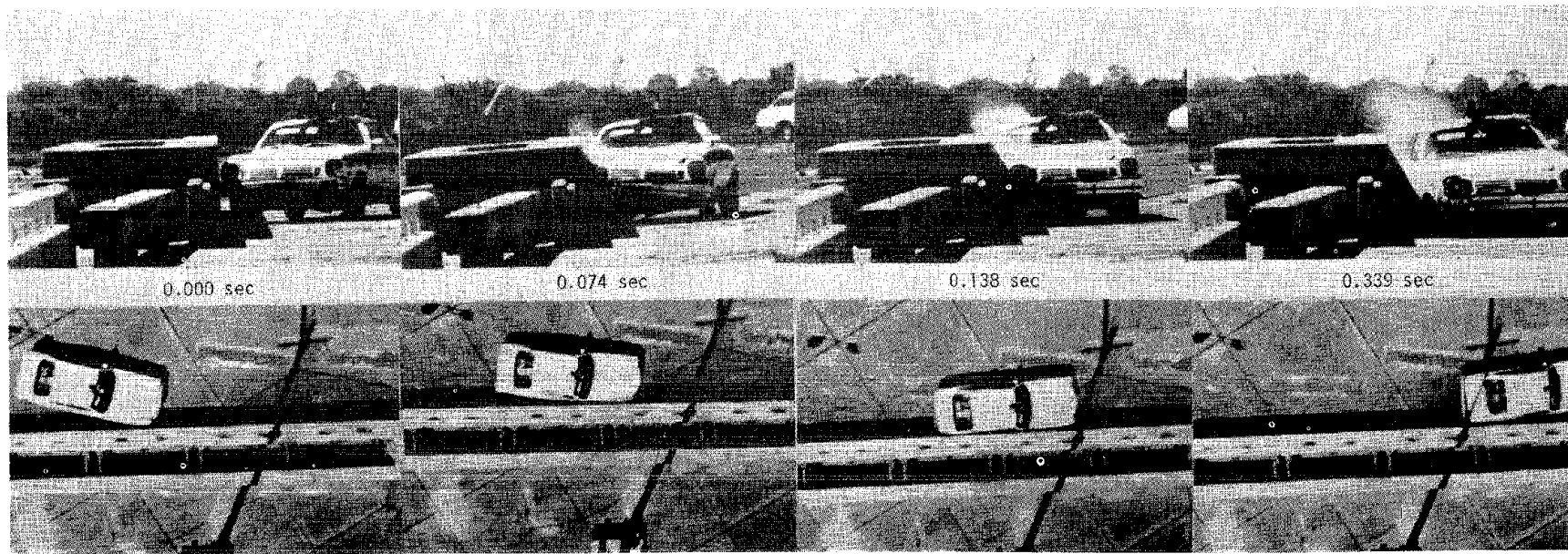
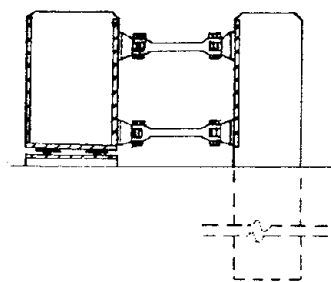
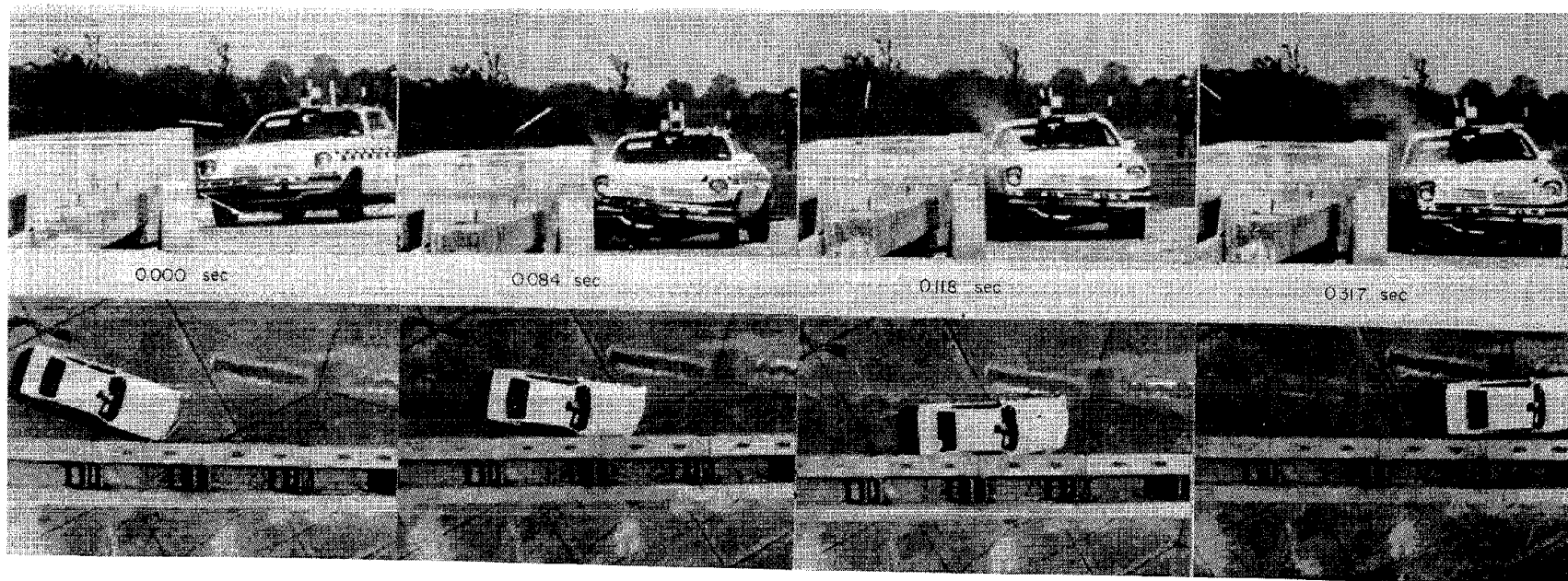


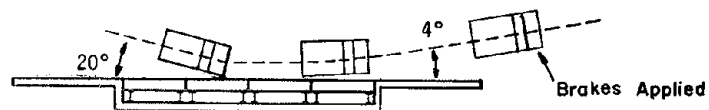
Figure 63. Summary of Results for Test 3451-30.



Figure 64. Photograph of Vehicle After Test 3451-30.



INSTRUMENTED WALL



Test No.	3451-31	Vehicle.	1975 Vega
Date	4/18/80	Vehicle Weight	1284 kg (2830 lb)
Instrumented Wall. . . .	Four Blocks; 3.084 m x 1.067 m x .457 m (10 ft x 3.5 ft x 1.5 ft)	(w/dummies, instr.)	
Instrumentation.	Four load cells and one accelerometer per block	Dummy Restraints	
Length of Installation . .	22.88 m (75.08 ft)	Driver	Lap Belt
Vehicle Damage		Passenger	No Restraint
TAD.	1RFQ3	Impact Speed	90.1 kph (56.0 mph)
SAE.	01RFEE2	Impact Angle	20.0 deg
		Exit Speed	73.7 kph (45.8 mph)
		Exit angle	4.0 deg
		Vehicle Acceleration	
		(Max. 0.050 sec avg)	
		Longitudinal	-3.6 g
		Transverse	8.1 g
		Vertical	1.8 g

Figure 65 . Summary of Results for Test 3451-31.

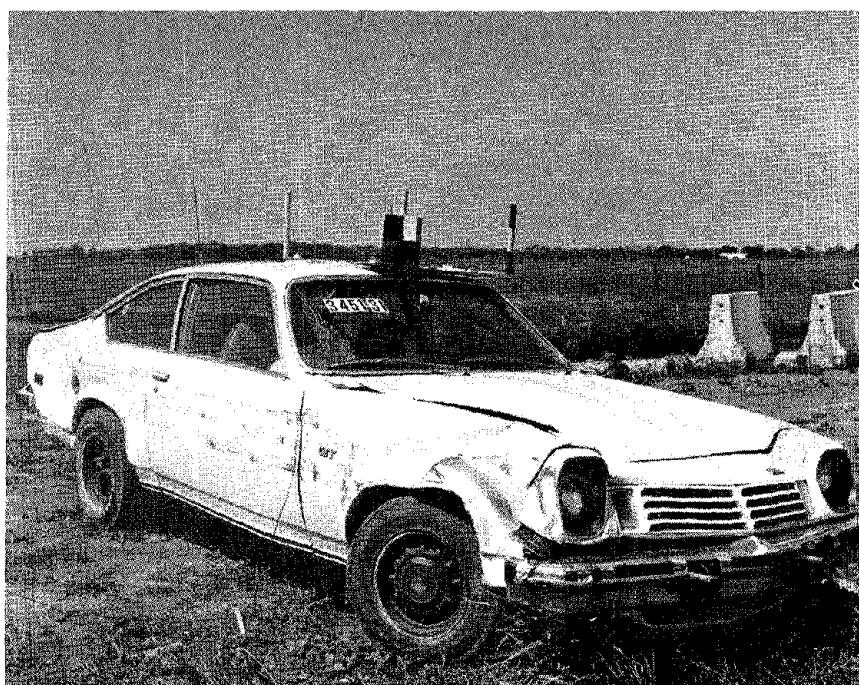
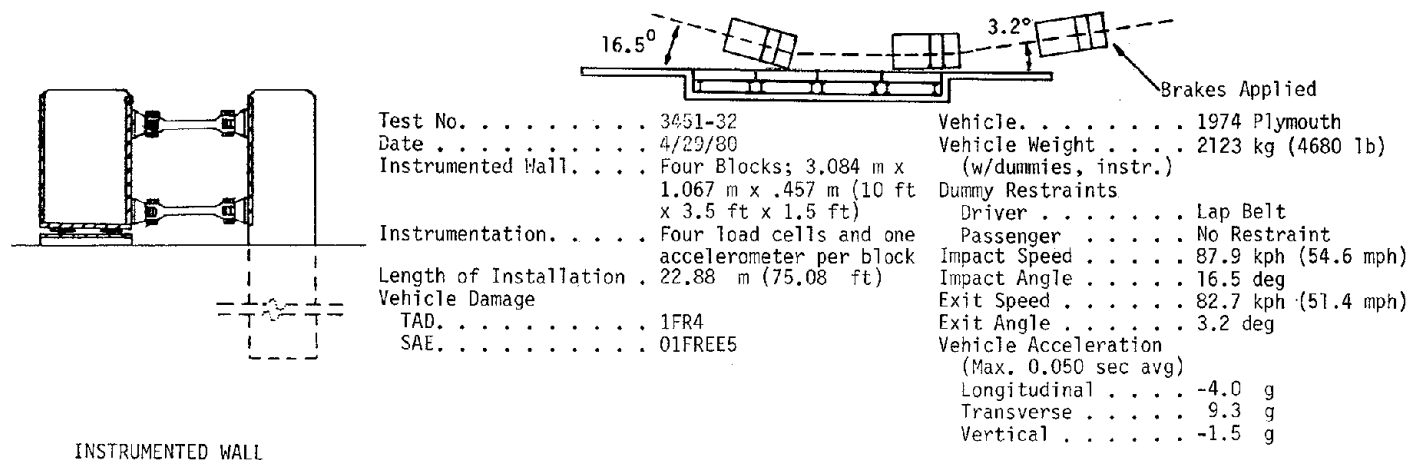
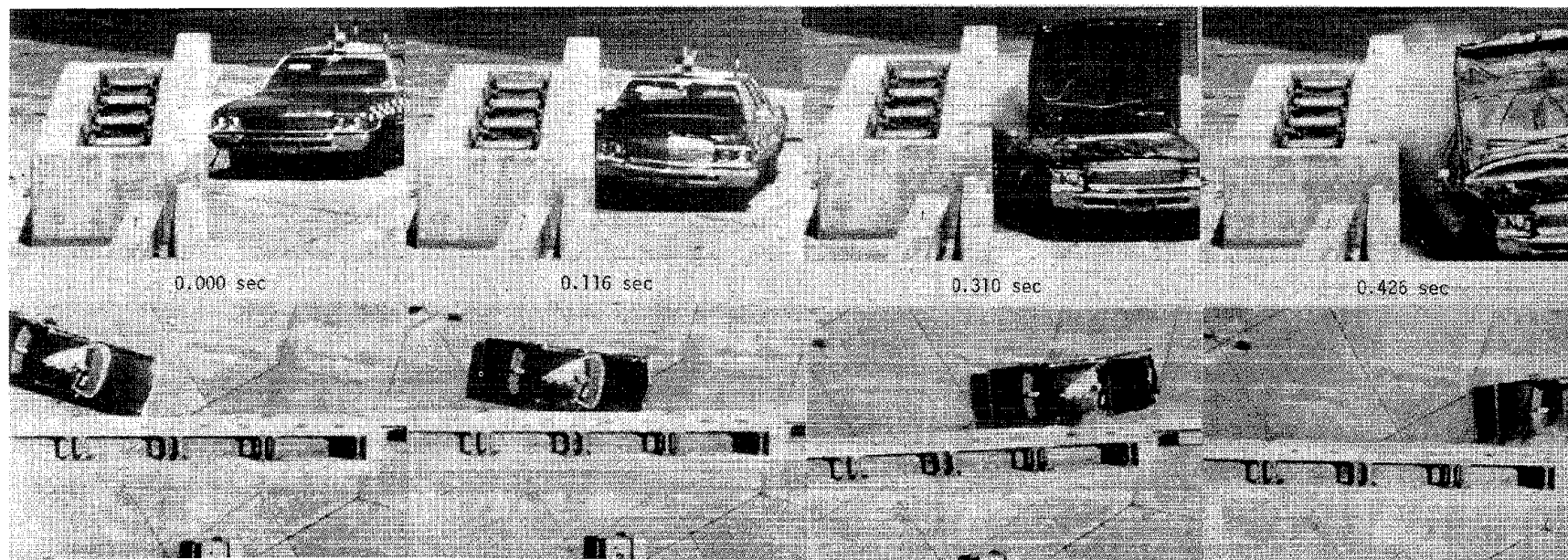


Figure 66. Photograph of Vehicle After Test 3451-31.



INSTRUMENTED WALL

Figure 67. Summary of Results for Test 3451-32.



Figure 68. Photograph of Vehicle After Test 3451-32.

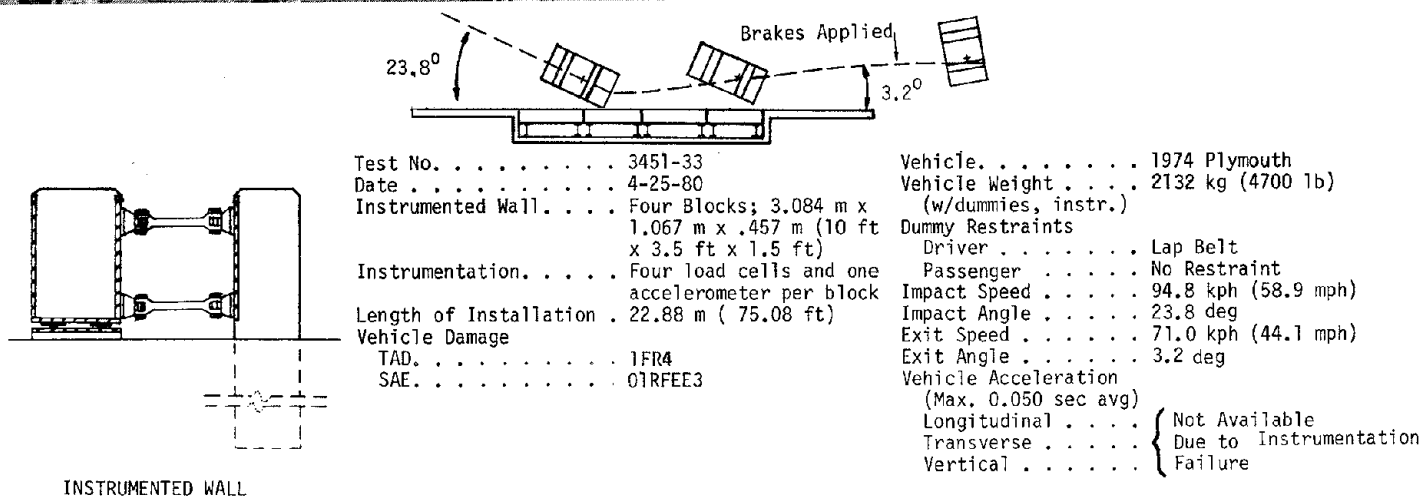
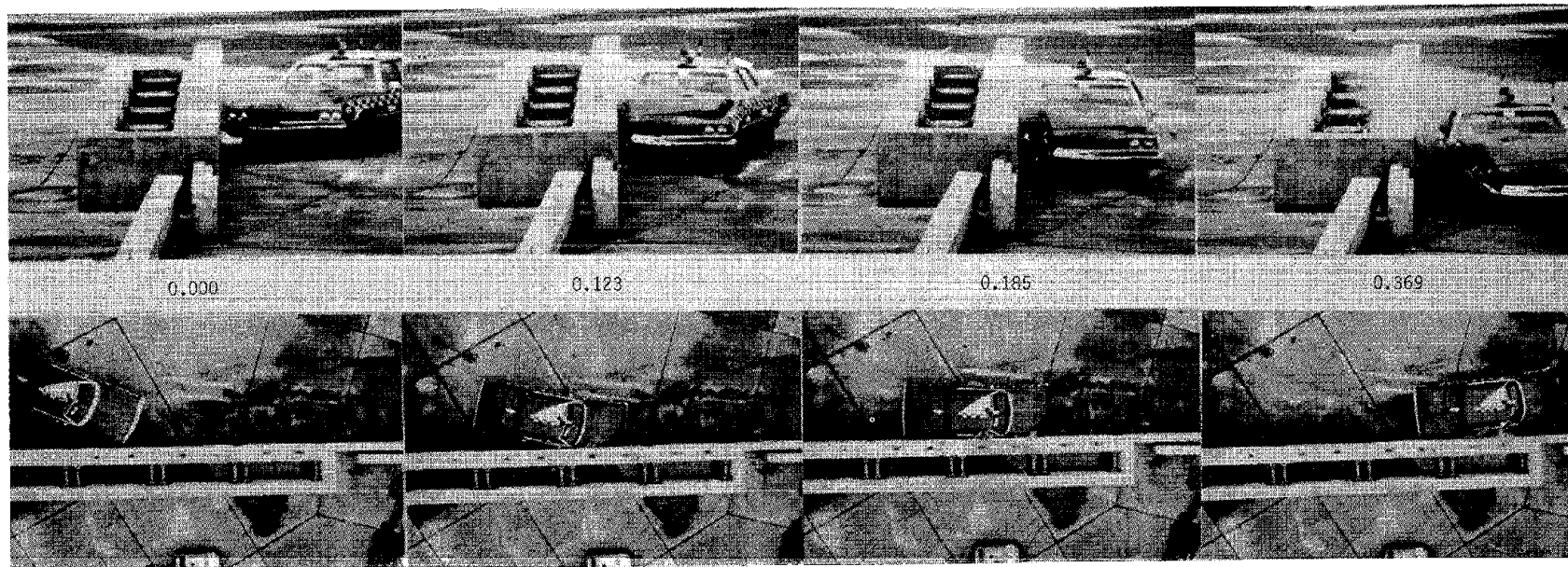


Figure 69 . Summary of Results for Test 3451-33.

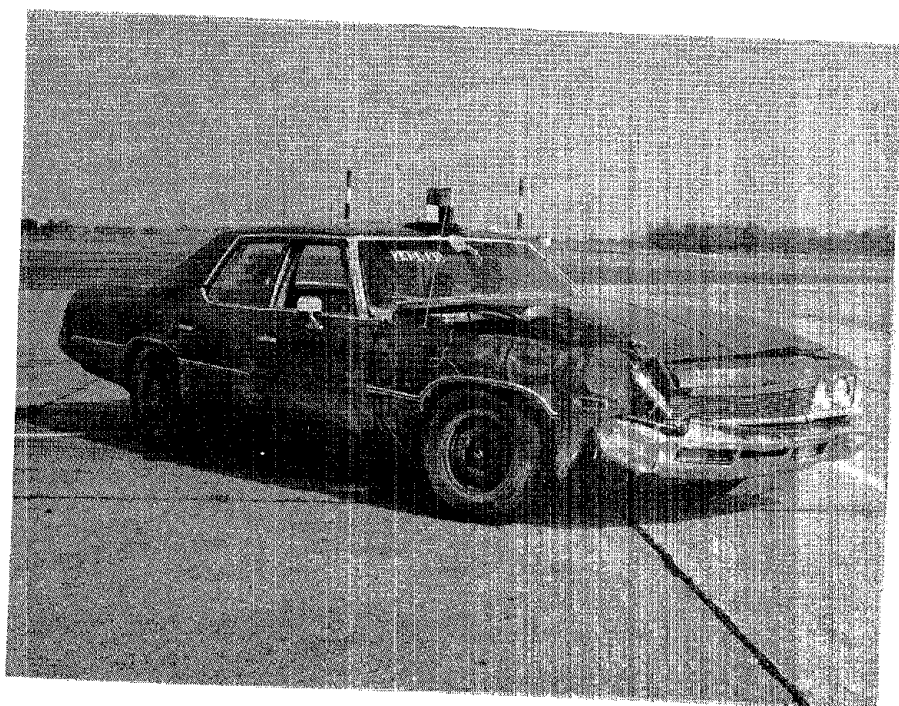


Figure 70. Photograph of Vehicle After Test 3451-33.

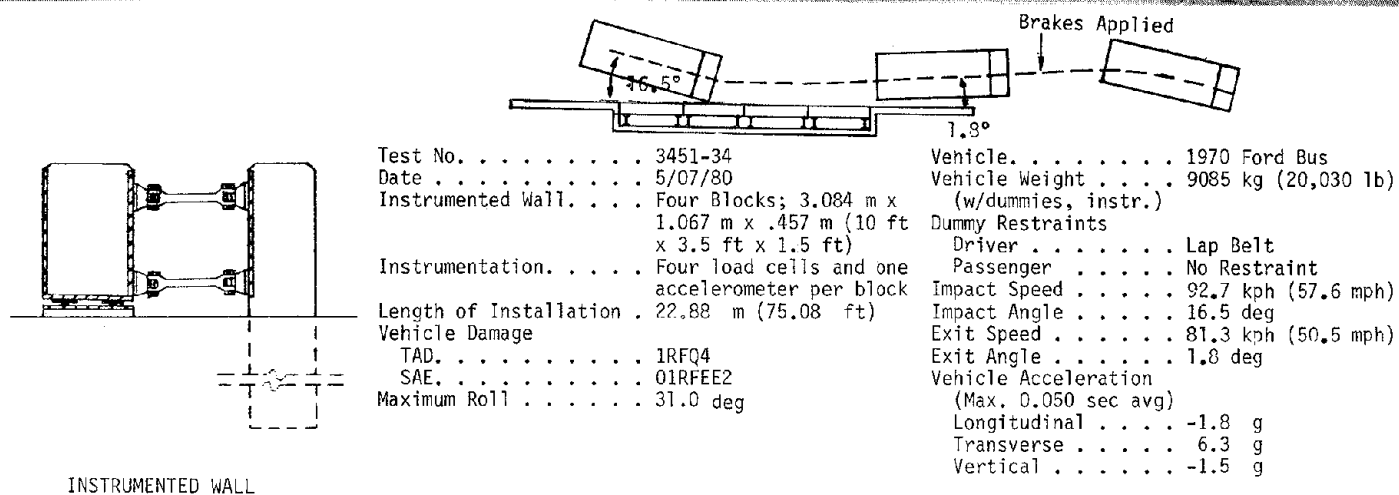
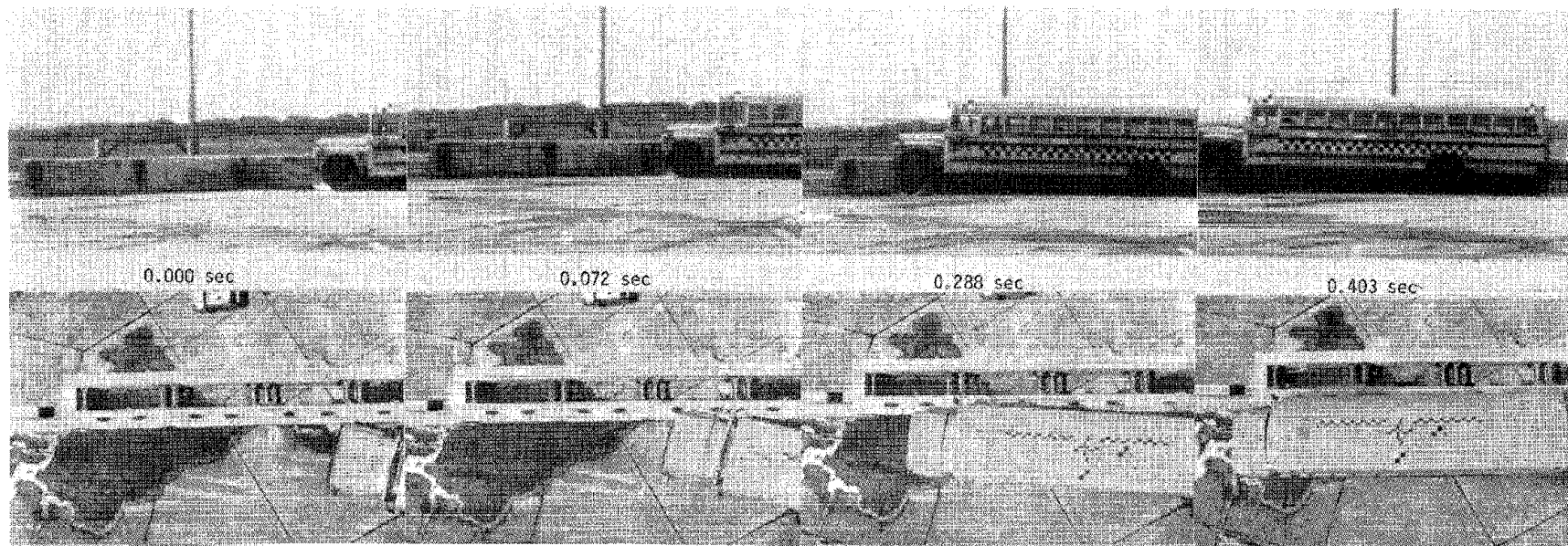


Figure 71. Summary of Results for Test 3451-34.



Figure 72. Photograph of Vehicle After Test 3451-34.

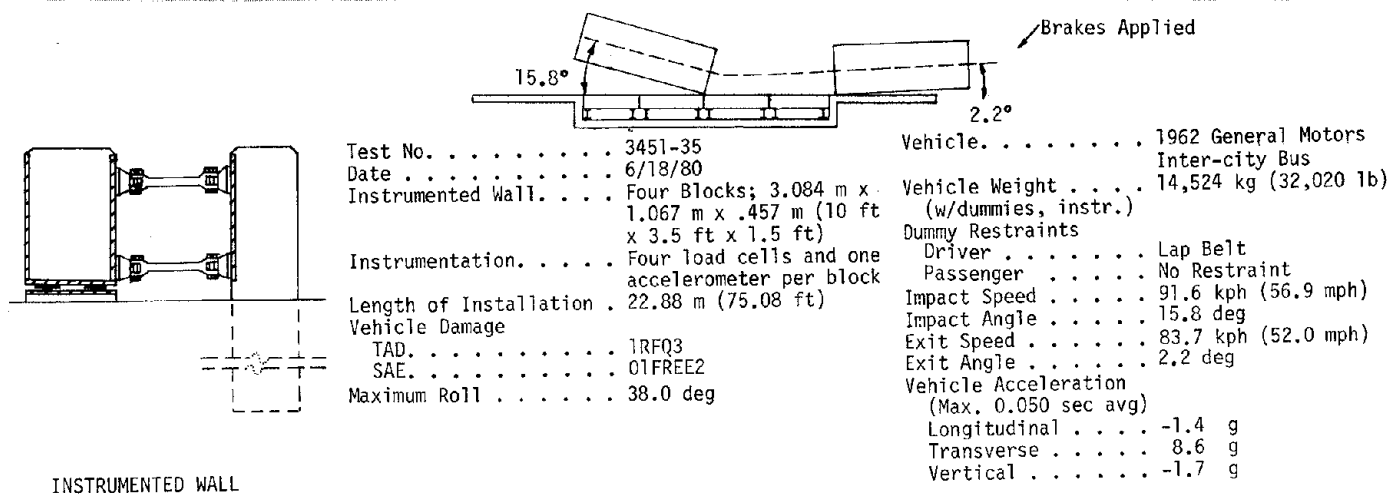
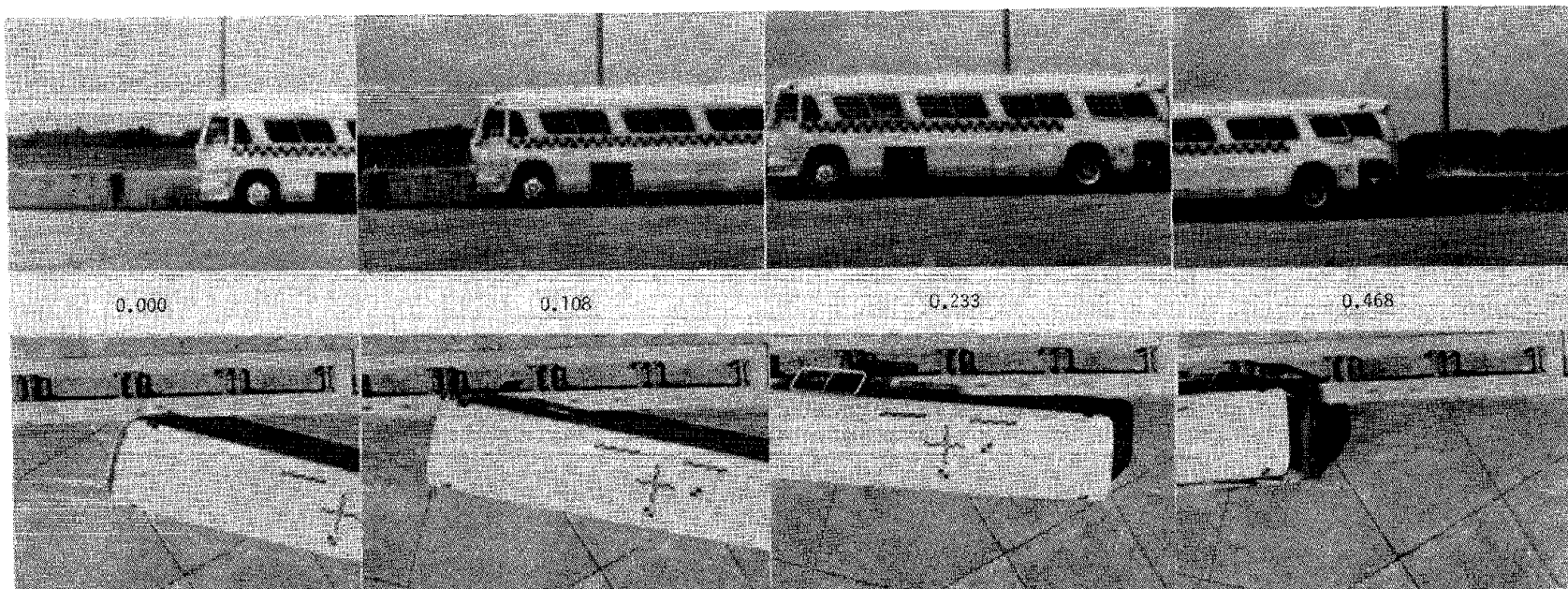
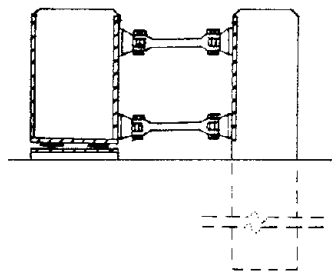
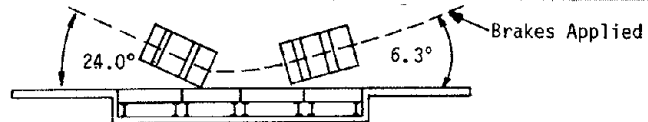
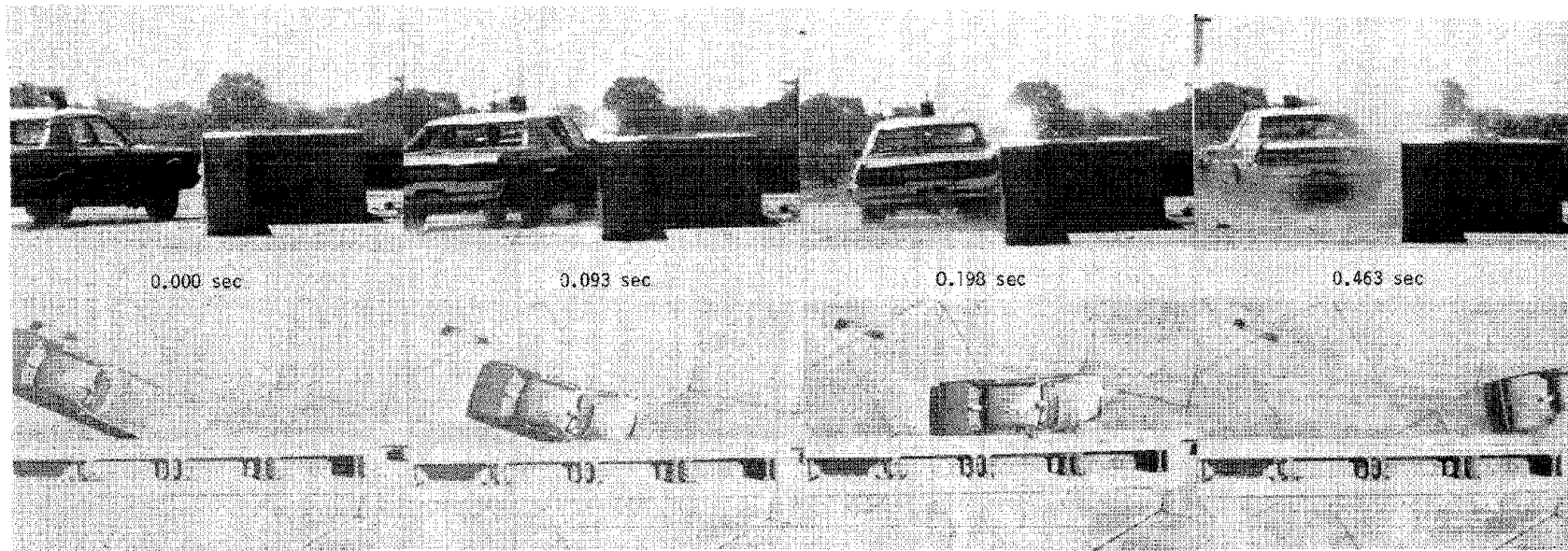


Figure 73. Summary of Results for Test 3451-35.



Figure 74. Photograph of Vehicle After Test 3451-35.



INSTRUMENTED WALL

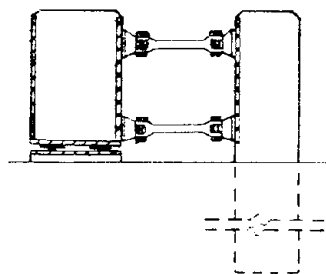
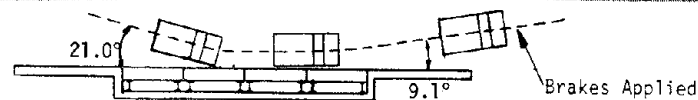
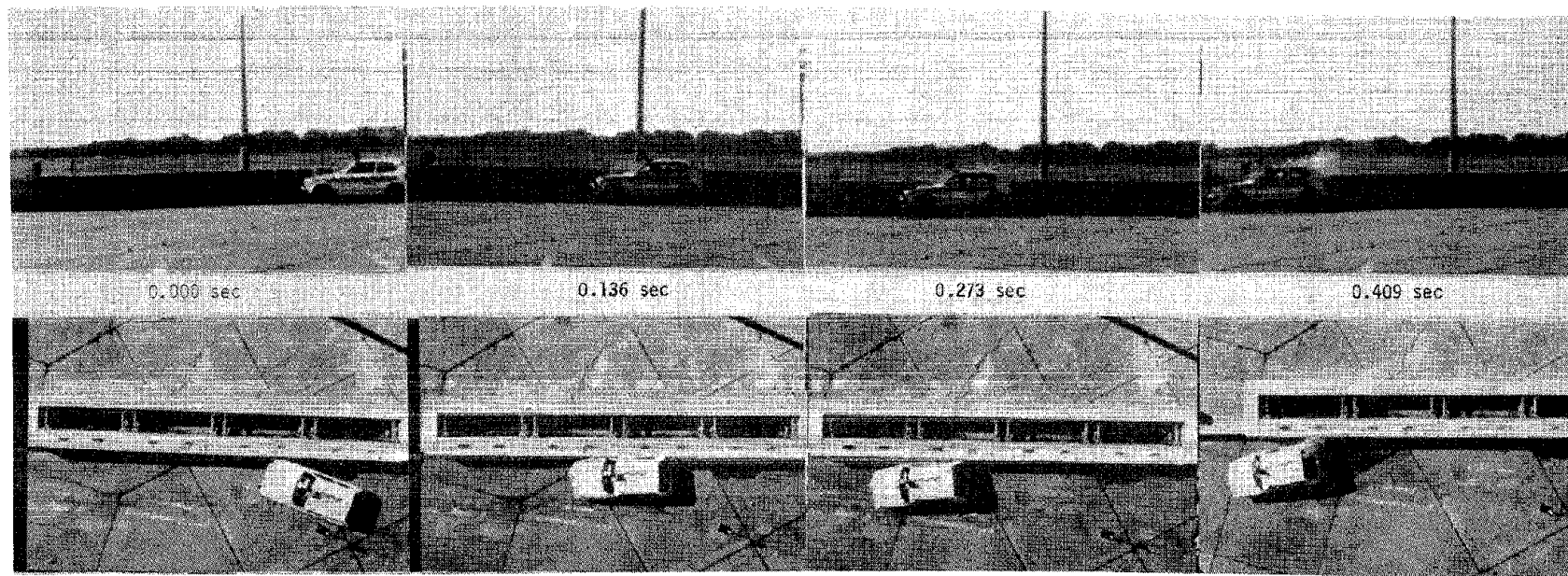
Test No. 3451-36
 Date 5-20-80
 Instrumented Wall. . . . Four Blocks; 3.084 m x
 1.067 m x .457 m (10 ft
 x 3.5 ft x 1.5 ft)
 Instrumentation. . . . Four load cells and one
 accelerometer per block
 Length of Installation . 22.88 m (75.08 ft)
 Vehicle Damage
 TAD. 1RFQ5
 SAE. 01RDAN9

Vehicle. 1975 Plymouth
 Vehicle Weight 2150 kg (4740 lb)
 (w/dummies, instr.)
 Dummy Restraints
 Driver Lap Belt
 Passenger No Restraint
 Impact Speed 96.2 kph (59.8 mph)
 Impact Angle 24.0 deg
 Exit Speed 68.2 kph (42.4 mph)
 Exit Angle 6.3 deg
 Vehicle Acceleration
 (Max. 0.050 sec avg)
 Longitudinal -9.1 g
 Transverse 15.4 g
 Vertical -2.3 g

Figure 75. Summary of Results for Test 3451-36.



Figure 76. Photograph of Vehicle After Test 3451-36.



INSTRUMENTED WALL

Test No.	3451-37	Vehicle.	1976 Honda
Date	10/24/80	Vehicle Weight	948 kg (2090 lb)
Instrumented Wall. . . .	Four Blocks; 3.084 m x 1.067 m x .457 m (10 ft x 3.5 ft x 1.5 ft)	(w/dummies, instr.)	
Instrumentation.	Four load cells and one accelerometer per block	Dummy Restraints . .	
Length of Installation . .	22.88 m (75.08 ft)	Driver	Lap Belt
Vehicle Damage		Passenger	No Restraint
TAD.	RD6	Impact Speed	94.1 kph (58.5 mph)
SAE.	RDW1	Impact Angle	21.0 deg
		Exit Speed	69.7 kph (43.3 mph)
		Exit Angle	9.1 deg
		Vehicle Acceleration	
		(Max. 0.050 sec avg)	
		Longitudinal	-6.5 g
		Transverse	13.1 g
		Vertical	-1.0 g

Figure 77. Summary of Results for Test 3451-37.



Figure 78. Photograph of Vehicle After Test 3451-37.

III. DESIGN GUIDELINES

General

Recommended guidelines for geometric requirements and collision forces (loads) for bridge railings are presented in this section. Detailed supporting material is presented in Appendix D. These guidelines address a wider range of vehicle size than is addressed in the current AASHTO bridge railing specifications (18) and should be considered in extending and revising the current specifications. If these recommendations are followed, bridge railing designs with improved crashworthiness will be achieved. However, the data available is limited and there is no guarantee that new railing designs will be completely adequate in all features. It is recommended that these guidelines be used in the development of improved bridge railing designs but that final acceptance of a design be based on performance demonstrated through full-scale crash tests.

Five in-service bridge railing designs were tested and evaluated in this study. A series of tests was also performed on a flat-faced, load-measuring, instrumented wall. All of the in-service railings were being used at the time the study was undertaken. Some were considered to meet strict interpretations of the AASHTO Specifications (17), but others were not. The most common discrepancy that was found had to do with geometrics not meeting the specifications. Some railing designs demonstrated unacceptable crashworthiness in automobile tests for which they were designed, even when geometrics met the specifications. Causes of this poor performance were identified and form a portion of the basis for these recommended design guidelines. Theoretical considerations, other crash tests (15, 16, 31, 32, 33) and a series of full-scale crash tests on a load measuring wall constitute the remaining basis.

A traffic railing must contain and smoothly redirect the selected design vehicles with minimum accelerations when tested under prescribed conditions of speed and approach angle. Railing features that must be considered in designing to meet these objectives are as follows:

1. Geometrics
2. Strength
3. Flexibility or Stiffness

Although each of these may be addressed somewhat independently, in the final analysis they all interact to determine performance of the railing. The approach taken here is to first describe acceptable geometrics that would be expected to result in "smooth" redirection of vehicles within the size range and impact conditions being considered. Once this is accomplished and the phenomenon being addressed is limited to acceptable "smooth" redirections, then the required strength and flexibility of the railing may be described.

Design Vehicles

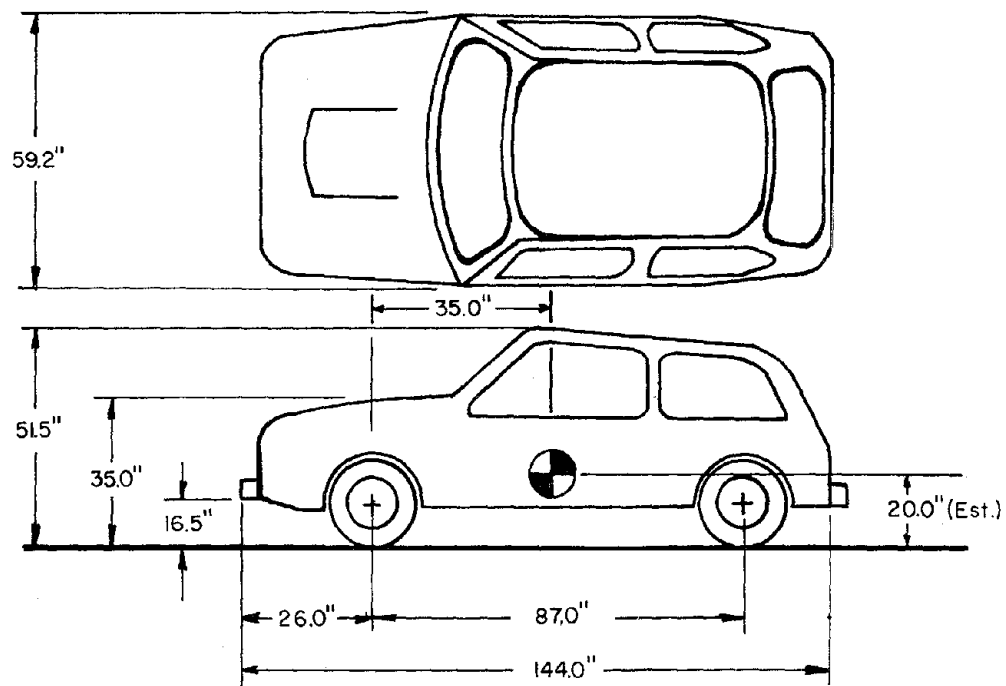
Vehicles studied in this program ranged in size from a 817 kg (1,800 lb) Honda Civic to a 14,528 kg (32,000 lb) intercity bus. However, a subpart of this vehicle spectrum may be addressed with the guidelines presented. Figures 79 through 83 present descriptions of vehicles used in this program and considered in development of design guidelines.

Geometrics of Railings

Geometric features necessary to describe an acceptable bridge railing system are:

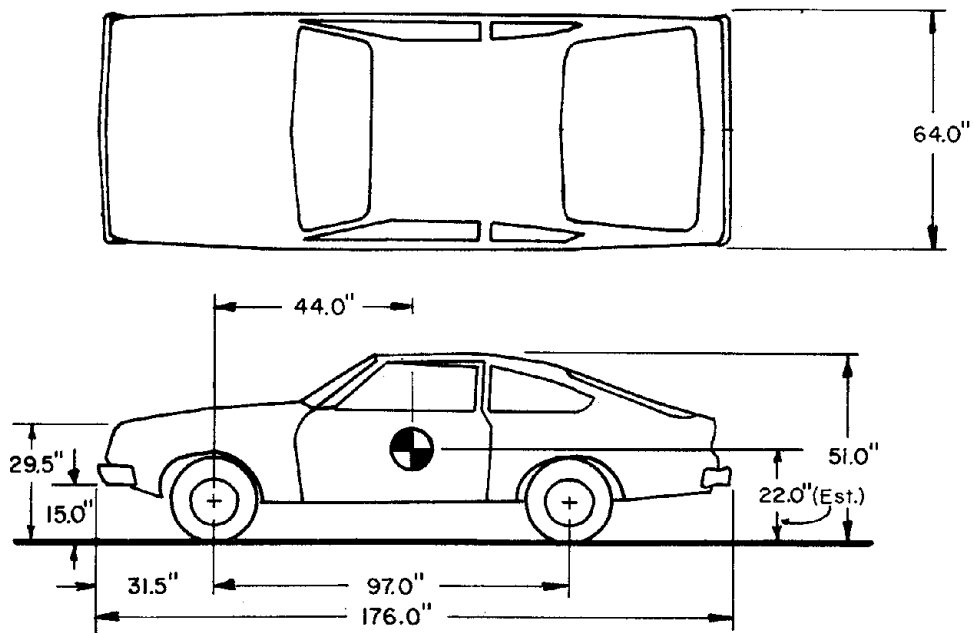
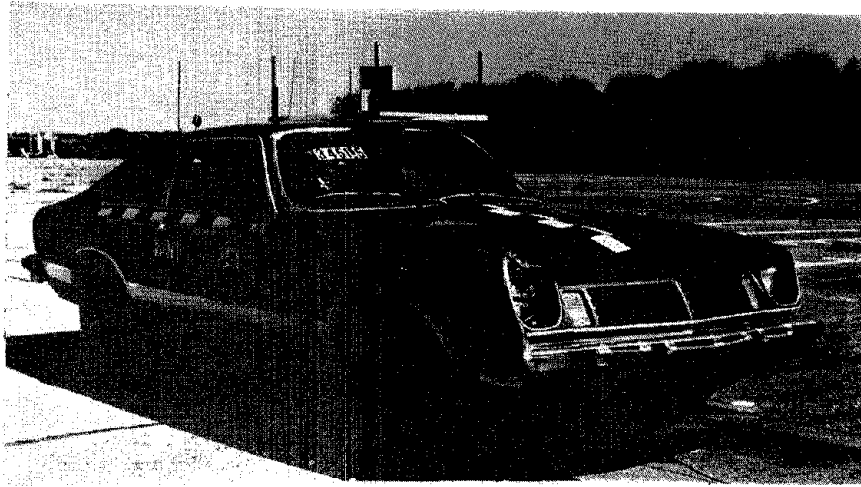
1. Effective Height of Railing
2. Presence and Location of Curbs
3. Vertical Openings/Post Setback/Frontal Area

Effective Height of Railing: A bridge railing must be high enough not only to prevent the vehicle from vaulting over or traversing the railing, but also to prevent the vehicle from rolling onto the railing. Historically, the height of a railing design has been thought of as being the overall height to the top extremity of the upper rail element measured from a referenced surface such as the top of the deck or safety walk. In beam and post systems particularly, and to some extent in concrete parapet railing systems, the resisting force provided by the railing is not at this upper extremity, but is somewhat lower. The resisting force provided by the top rail element in a beam and post system is probably closer to the centroid of the rail element. For a concrete parapet or wall, the resisting force may be within 0.05 to 0.08 m (2 to 3 in.) of the top extremity. Therefore, when one considers the influence of railing height on vehicle roll behavior, the effective height rather than the total geometric height should be considered. The effective height is defined as



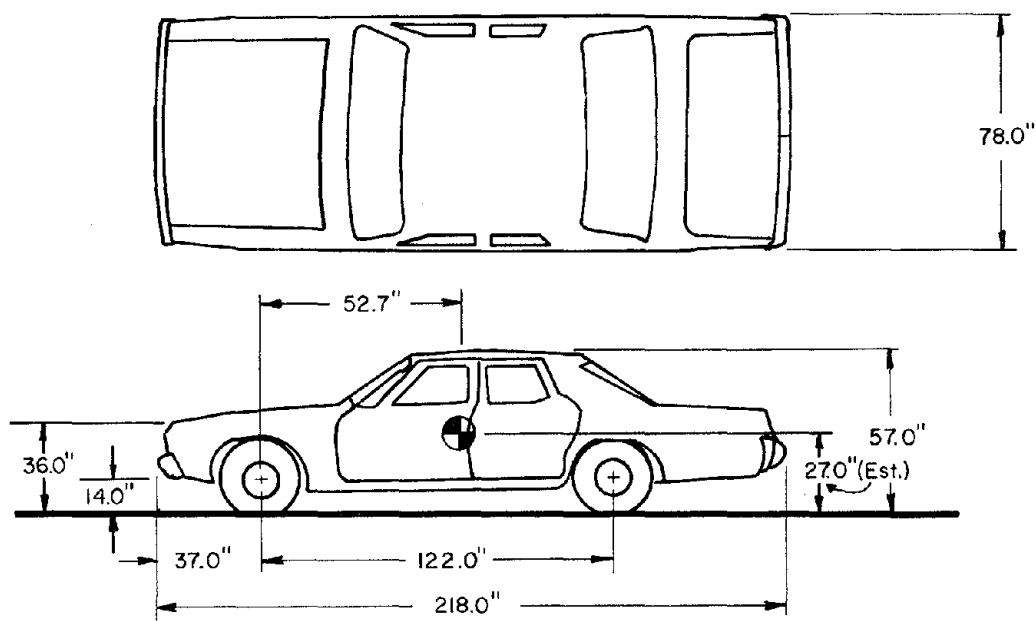
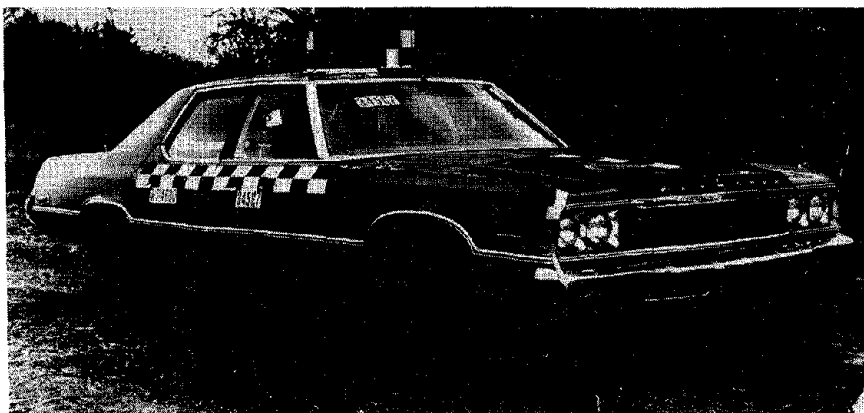
Test Inertial Mass:	1,620 lb	1 lb = 0.454 kg
Dummy Mass:	330 lb	1 in. = 0.025 m
Loose Ballast Mass:	0 lb	
Gross Static Mass:	1,950 lb	
Wheel Diameter:	21.5 in.	
Bumper Height:	16.5 in. to bottom	20.5 in. to top

Figure 79. 1974 Honda.



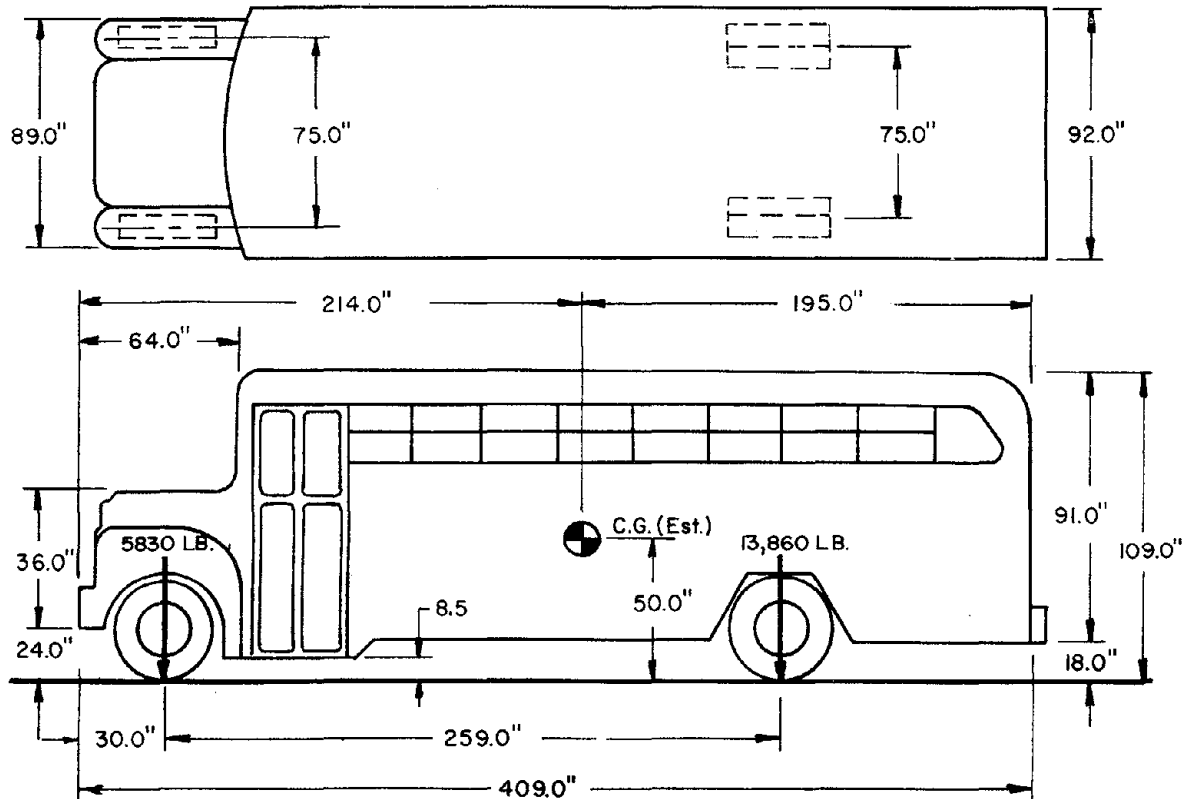
Test Inertial Mass:	2,450 lb	1 lb = 0.454 kg
Dummy Mass:	330 lb	1 in. = 0.025 m
Loose Ballast Mass:	0 lb	
Gross Static Mass:	2,780 lb	
Wheel Diameter:	22.5 in.	
Bumper Height:	15.0 in. to bottom	20.0 in. to top

Figure 80. 1974 Vega.



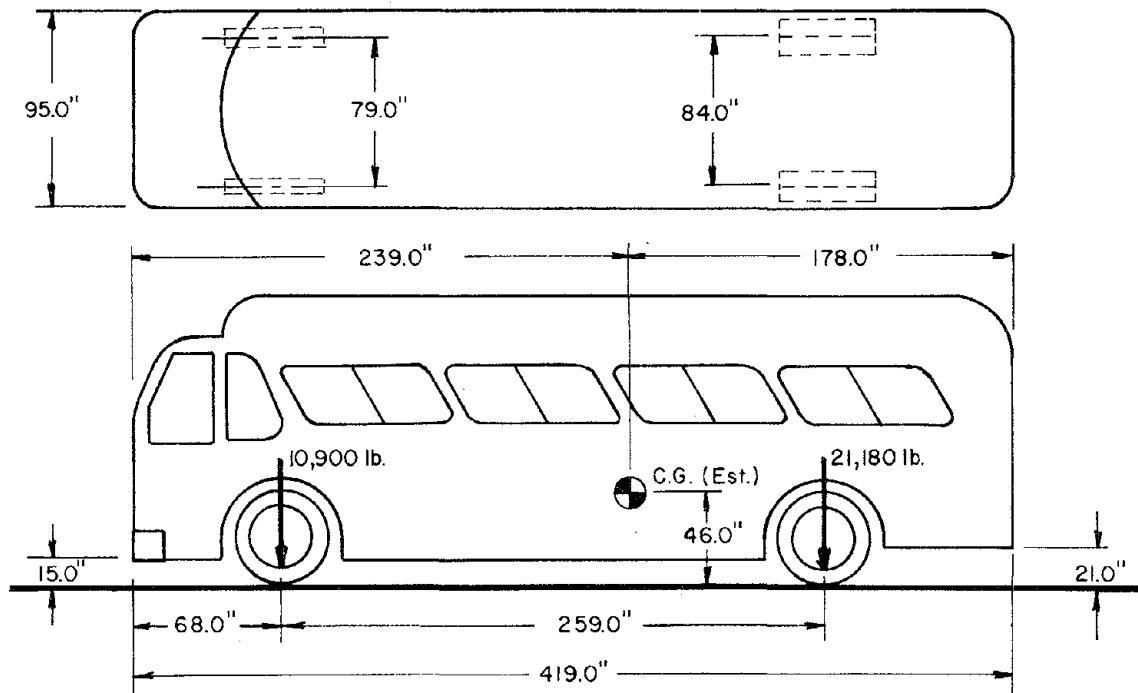
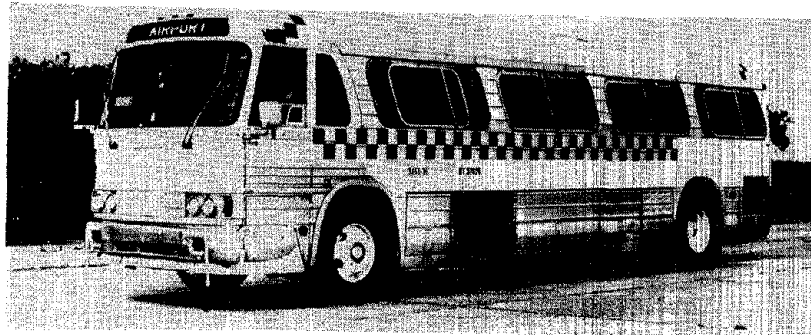
Test Inertial Mass:	4,300 lb	1 lb = 0.454 kg
Dummy Mass:	330 lb	1 in. = 0.025 m
Loose Ballast Mass:	0 lb	
Gross Static Mass:	4,630 lb	
Wheel Diameter:	27.5 in.	
Bumper Height:	14 in. to bottom	21 in. to top

Figure 81 . 1974-76 Plymouth Fury.



Test Inertial Mass:	12,800 lb	1 lb = 0.454 kg
Dummy Mass:	495 lb	1 in. = 0.025 m
Loose Ballast Mass:	6,505 lb	
Gross Static Mass:	19,800 lb	
Wheel Diameter:	37 in.	
Bumper Height:	24.0 in. to bottom	32.0 in. to top

Figure 82 . 1970 Ford Sixty-Six Passenger School Bus.



Test Inertial Mass:	21,000 lb	1 lb = 0.454 kg
Dummy Mass:	495 lb	1 in. = 0.025 m
Loose Ballast Mass:	10,505 lb	
Gross Static Mass:	32,000 lb	
Wheel Diameter:	42 in.	
Bumper Height:	15.0 in. to bottom	27.0 in. to top

Figure 83. 1962 GM PD4106 Coach.

the distance from the pavement surface to the centroid of the resisting force provided by the railing system. The effective height is not a unique value for a given railing design but is a function of the magnitude of the resisting force (6).

Required effective heights of railings to prevent rollover of the vehicle are presented in Table 7. These recommended effective heights are based on theoretical considerations, measured heights of resultant forces from tests on the instrumented wall, results of tests on in-service railings reported herein, and results of tests reported by others. Details of these considerations are contained in Appendix D.

Presence and Location of Curbs: Two railing systems with curbs, the New Hampshire Two-Bar Aluminum Railing with a protruding curb and the Colorado Type 5 Railing with a flush curb, were tested in this study. Also, significant test data and HVOSM simulation results involving interaction of automobiles with curbs are reported in NCHRP Report 150 (14). That report demonstrates that curbs of the type and height studied therein offer very little redirection capability except at very small encroachment angles. A 0.15 m (6 in.) high, type C curb, for example, did not provide redirection of automobiles at speeds above 97 kph (60 mph) and encroachment angles above 5 deg.

In testing with automobiles on the New Hampshire railing reported herein, impact angles were 15 and 20 deg. In all tests, the automobile front wheel traversed the curb and interacted with the metal portion of the railing. The tire was deflated by the curb and the automobile continued forward with the lower extremity of the metal portion of the wheel rolling on the top surface of the curb. This left the wheel aligned with the 0.36 m (14 in.) vertical opening between the top of the curb and the lower metal rail element. The automobile then continued forward with the bumper and wheel snagging on the next downstream post. In 15 deg tests with the Honda and Vega, snagging was more severe than in the 20 deg test and the railing suffered structural damage. Snagging and structural damage also occurred in the 15 deg test with the 2,043 kg (4,500 lb) automobile.

Similar, although less severe, behavior occurred in automobile tests on the Colorado Type 5 railing. In 15 deg tests with a Vega and a Plymouth, the curb deflated the front tire and the vehicle penetrated the

Table 7 . Required Effective Height of Railing
to Prevent Rollover of Vehicle.

VEHICLE	TEST CONDITIONS	REQUIRED MINIMUM EFFECTIVE RAILING HEIGHT (in.)
Automobiles 1,800 to 4,500 lb	60 mph and up to 25 deg	24
School Bus 20,000 lb	60 mph/15 deg	34
Intercity Bus 32,000 lb	60 mph/15 deg	30

1 lb = 0.454 kg
1 mph = 1.609 kph
1 in. = 0.025 m

0.38 m (15 in.) opening between the top of the curb and the bottom of the metal rail element. This effect was more pronounced in the 25 deg test with a 2,043 kg (4,500 lb) automobile.

Although these two railing designs have other undesirable geometrics which contributed to their poor performance, the curbs, especially the protruding curb in the New Hampshire design, were the cause of much damage to the test vehicle. Again, curbs offer little redirection capability. They are extremely rigid and their positioning serves to promote damage to an automobile wheel. Curbs, either flush or protruding, should not be used as a part of a bridge railing system.

On bridge structures where it is necessary to contain runoff water, curbs that are setback behind the traffic face of the railing (for beam and post railing systems) or concrete parapet type railings should be considered. If curbs are used in this position with beam and post system, their presence should be ignored when determining the vertical clear opening between the deck and lower railing.

Some railing designs, such as the North Carolina railing, make use of a concrete parapet (perhaps 0.46 m (18 in.) high) with a metal rail mounted on top of it. While the 0.46 m (18-in.) concrete parapet may be acceptable for automobiles (no automobile tests were conducted on the North Carolina railing) it is not recommended for larger vehicles such as school and intercity buses until further testing is performed. There are two reasons for this. The 0.46 m (18-in.) dimension is thought to be inadequate to prevent climbing of the larger diameter wheels. The nondeflecting curb or parapet in combination with a deflecting top rail element promotes roll of the larger vehicle. If concrete parapets with metal rails on top are designed for these large buses and trucks, the concrete parapet should be at least 0.69 m (27 in.) high to prevent climb of the large diameter wheels.

Vertical Openings/Post Setback/Frontal Area: Railing systems tested and reported herein that provide data on the influence of vertical openings and post setback on snagging are the Texas T101, Indiana 5A, and the modified Indiana 5A. Tests conducted on Texas HPR Study 230 (7) also provide data relative to this question.

Based on information gained from this group of tests, it was hypoth-

esized that railing design features which influence snagging of a vehicle wheel on railing posts are frontal area of the railing elements, heights of the vertical opening, post setback, and impact angle. In systems with multiple rail elements, location of the upper rail elements may also influence this relationship.

Vehicle geometrics and the interaction of vehicles with railings in tests on in-service railings were analyzed to develop recommended geometrics to preclude snagging. In tests where snagging occurred, the amount of overlap of the vehicle wheel on posts was observed to arrive at required post setback distances for acceptable performance. These values are presented in Table 8 and Figure 84.

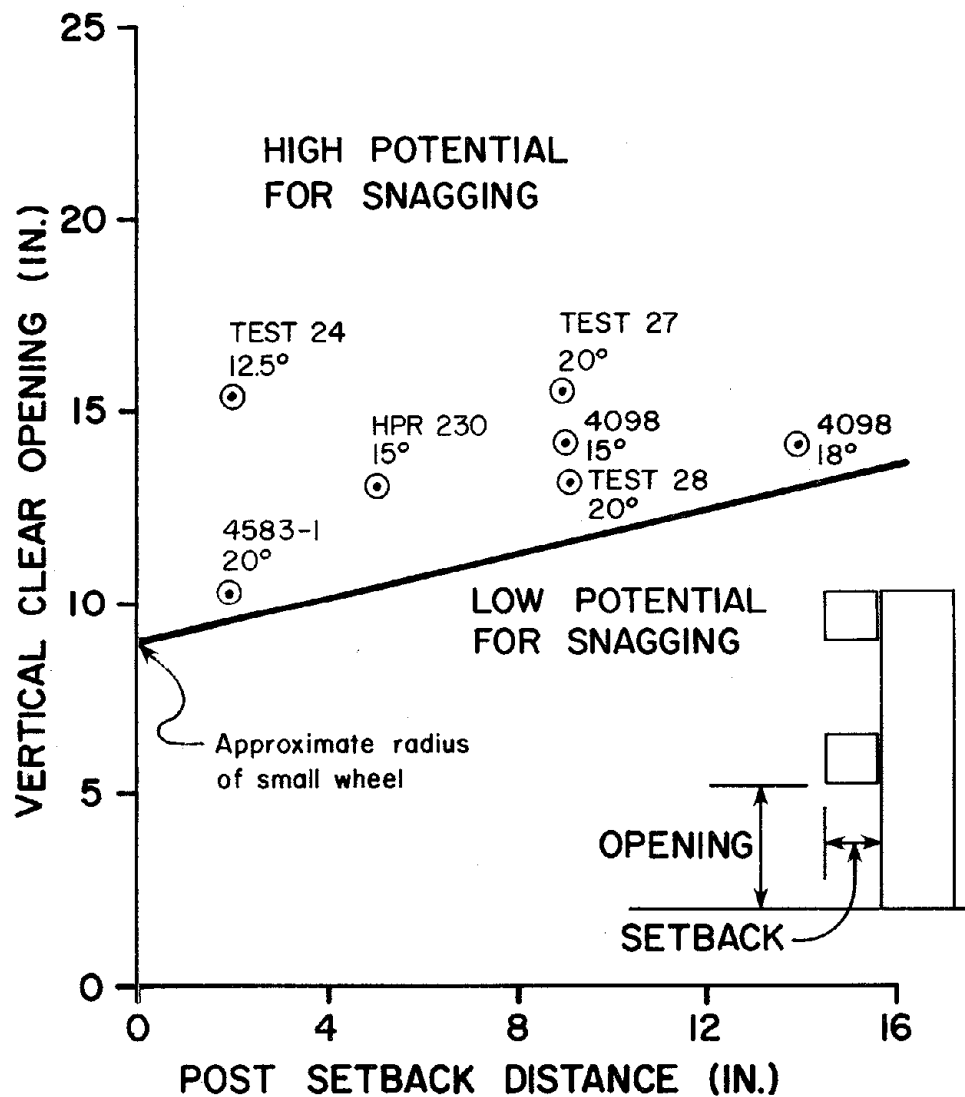
This figure shows the vertical clear opening that may be acceptable for a given post setback distance. A limiting line on the conservative side of the data has been drawn and is recommended for guidance in establishing geometry that will minimize or preclude snagging of front wheels on small automobiles such as a Honda Civic when impacting nominally rigid railing systems. This relationship should not be applied to weak post or breakaway post systems.

Close study of the test data presented herein leads one to observe that deep rail elements such as the W-section, the aluminum Tru-beam, or the flat concrete face on the HPR 230 railing generally offer better redirection capability than do the thinner rail elements. This is due, in most part, to the fact that the thinner rail elements "cut into" the automobile more than the deep beams do. The thinner rail elements can be made to perform successfully, but much larger post setback distances are required. In Test 28, (Modified Indiana 5A Railing) the Honda wheel underrode the lower rail element and at the same time the bumper overrode the rail element. It is desirable for one or more elements to virtually block the space from about 0.23 or 0.25 m (9 or 10 in.) above the deck to about 0.56 m (22 in.) above the deck in order to adequately prevent partial penetration (and snagging) of automobiles with various bumper and front wheel geometrics. However, it is recognized that innovative railing designs which do not meet these geometric guidelines may be developed and should be considered acceptable if performance is demonstrated through full-scale crash tests.

Table 8. Post Setback Distance Required to Prevent Snagging.

Test Number	Automobile	Impact Angle (deg)	Railing Design	Height of Opening (in.)	Required Post Setback (in.)
3451-24	Honda	12.5	Indiana 5A	15 3/8	2
3451-27	Honda	20.0	Indiana 5A	15 3/8	9
3451-25	Vega	19.5	Indiana 5A	15 3/8	5
3451-28	Honda	19.0	Modified Indiana 5A	13	9
	Honda	15.0	HPR 230	13	5
3451-5	Vega	15.0	T101	15	5 1/2 or less
4098-4	Honda	15.0	Thrie Beam Guardrail	14	9
4098-5	Honda	18.0	Thrie Beam Guardrail	14	14
4583-1	Honda	20.0	Aluminum Tru-beam	10 1/4	2

1 in. = 0.0254 m



1 in. = 0.0254 m
1 mph = 1.609 kph

Figure 84. Influence of Vertical Clear Opening and Post Setback on Potential for Snagging of Honda Civic Front Wheel for Test Conditions up to 60 mph and 20 deg.

Strength of Railing

Lateral forces imposed by various test vehicles at various impact angles were measured in the series of tests on the instrumented wall and are reported in Appendix C. Data from these tests were analyzed to determine magnitudes, locations of resultants and distributions of forces. The force-time trace from each test showed that two distinct impacts (or impulses) occurred during each test. The initial impulse occurred when the front corner of the vehicle was in contact with the wall and the vehicle was being redirected. The final impulse occurred when the rear of the vehicle swung against the wall. Each of these impulses were analyzed to determine the highest 0.050 sec average force. The vertical position of the centroid of the force was then determined at corresponding time intervals. Results of these analyses are presented in Table 9.

This data for the heavy car test at 25 deg, the school bus test at 15 deg and the intercity bus test at 15 deg are presented in Figures 85 through 93. It is noted that the highest 0.050 sec average lateral force imposed in the heavy car test at 25 deg was 266.4 kN (59.9 kips). This test has been considered to be an upper limit strength test for railing systems designed in accordance with the AASHTO specifications (17) using allowable strength design procedures and a 44 kN (10 kip) static service load. The resultant center of the force imposed in this test was about 0.56 m (22 in.) above the reference surface (Figure 87). Many of the railing systems that have been properly designed by AASHTO specifications have been found to meet the strength requirements of a 2,043 kg/97 kph/25 deg (4,500 lb/60 mph/25 deg test).

Data from a school bus test are presented in Figures 88 through 90. It is noted that the highest 0.050 sec average lateral force imposed during initial impact was 283.3 kN (63.7 kips) which is only slightly higher than that imposed by the heavy automobile. The school bus imposed an even higher force of 328.3 kN (73.8 kips) during the second impact when the rear of the bus hit the railing; however, the load was distributed over a longer length of railing. The resultant center of the force was about 0.84 m (33 in.) above the reference surface.

Comparison of the bus test data with the automobile test data provides an explanation of the fact that many current railing designs have suffic-

Table 9 . Summary of Data From Instrumented Wall Test Series.

Test Number	Test Conditions	Initial Impact				Final Impact			
		Force (kips)	X dist. (in.)	Y dist. (in.)	Time (sec)	Force (kips)	X dist. (in.)	Y dist. (in.)	Time (sec)
3451-29	Honda 1,970/59.0/15.5	18.4	217	16.0	.034-.084	8.4	214	18.7	.113-.163
3451-30	Vega 2,800/58.3/14.8	18.5	225	17.7	.019-.069	13.9	227	16.0	.101-.151
3451-31	Vega 2,830/56.0/20.0	22.0	225	20.0	.034-.084	22.5	231	21.0	.119-.169
3451-32	Plymouth 4,680/54.6/16.5	52.5	260	21.2	.054-.104	28.3	252	24.1	.154-.204
3451-34	School Bus 20,030/57.6/16.5	63.7	203	28.5	.110-.160	73.8	153	33.0	.335-.385
3451-35	Inter-City Bus 32,020/56.9/15.8	85.0	174	26.3	.041-.091	211.0	171	28.0	.331-.381
3451-36	Plymouth 4,740/59.8/24.0	59.9	125	21.9	.047-.097	28.3	125	22.5	.158-.208
3451-37	Honda 2,090/58.5/21.0	21.1	232	18.8	.036-.086	13.1	231	20.7	.103-.153

1 lb = 0.454 kg / 1 mph = 1.609 kph

1 kip = 4.448 kN / 1 in. = 0.025 m

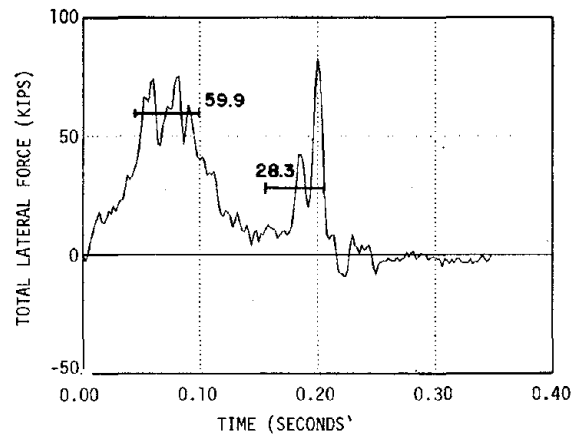


Figure 85. Total Lateral Force Measured by Wall Transducers on Wall for Test 3451-36.

2,150 kg/96.2 kph/24.0 deg
(4,740 lb/59.8 mph/24.0 deg)

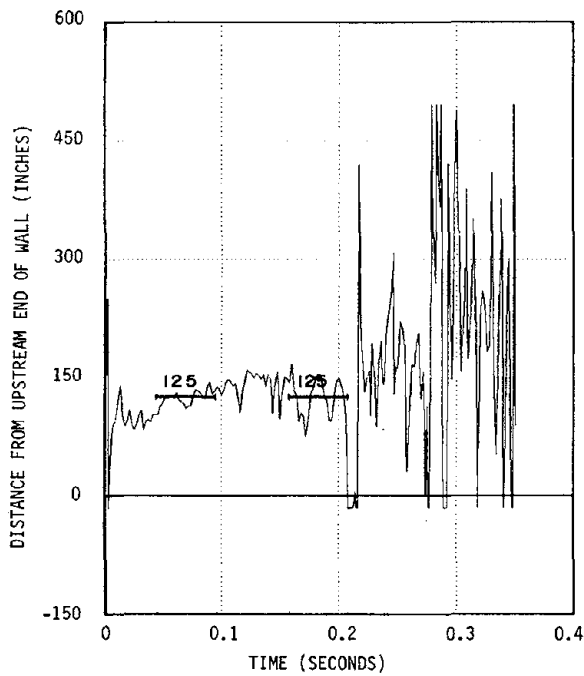


Figure 86. Horizontal Position of Total Lateral Force on the Wall for Test 3451-36.

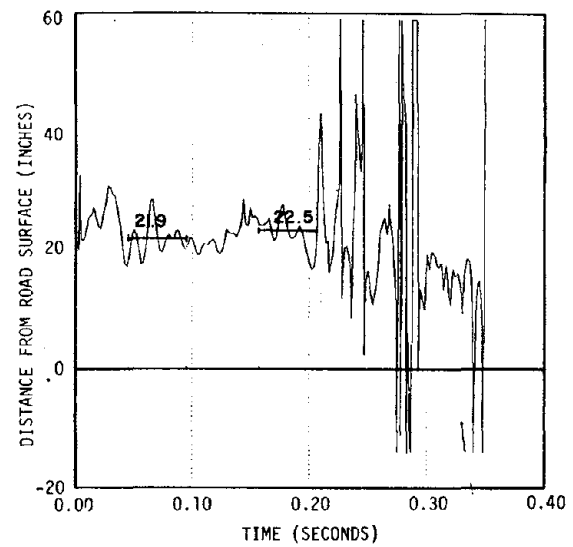


Figure 87. Vertical Position of Total Lateral Force on the Wall for Test 3451-36.

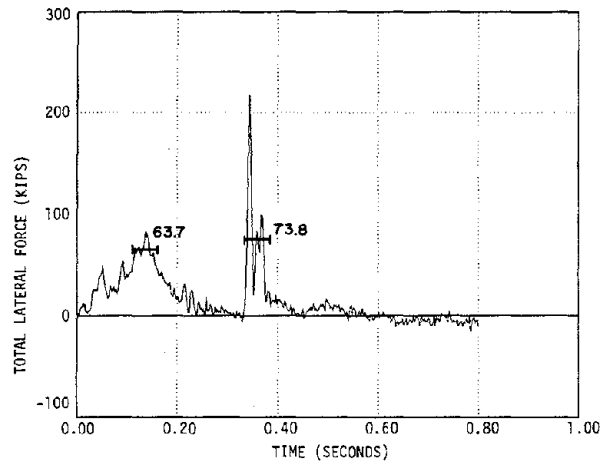


Figure 88. Total Lateral Force Measured by Wall Transducers on Wall for Test 3451-34.

9,085 kg/92.7 kph/16.5 deg
(20,030 lb/57.6 mph/16.5 deg)

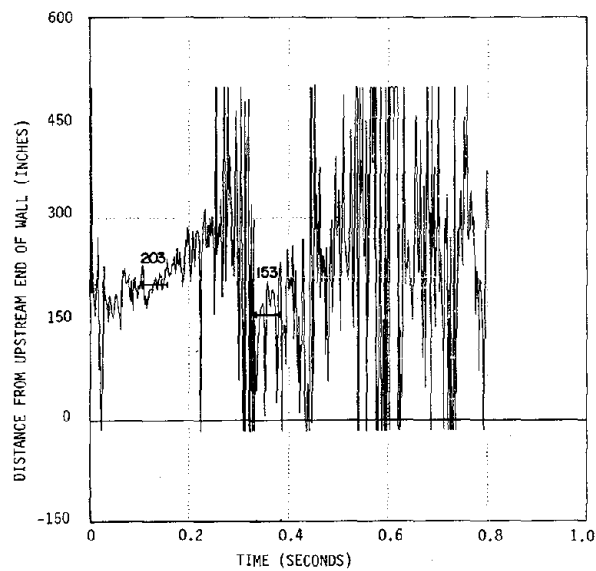


Figure 89. Horizontal Position of Total Lateral Force on the Wall for Test 3451-34.

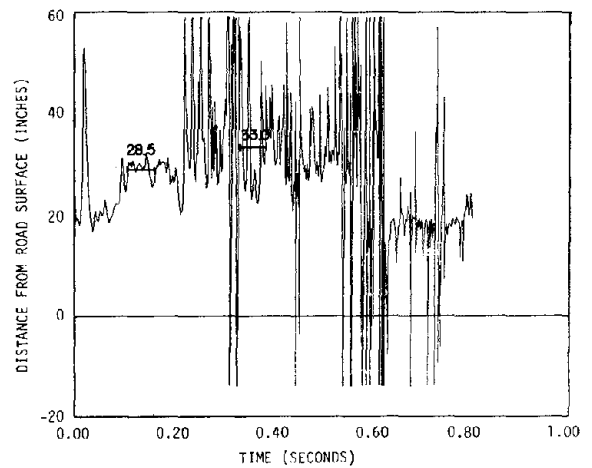


Figure 90. Vertical Position of Total Lateral Force on the Wall for Test 3451-34.

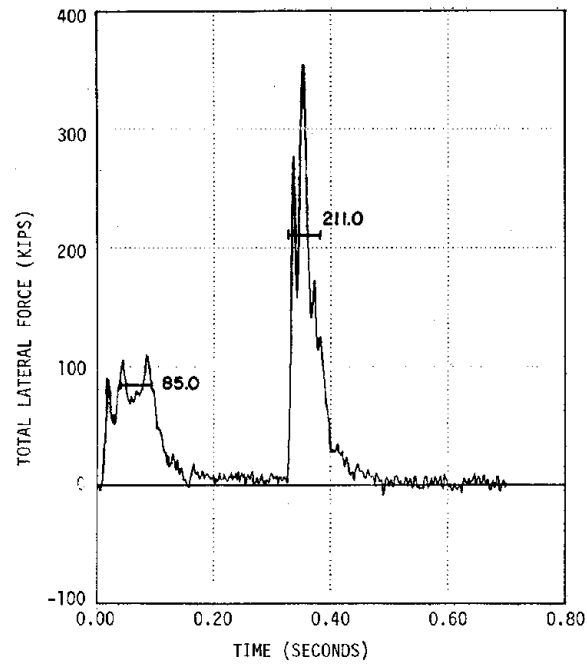


Figure 91. Total Lateral Force Measured by Wall Transducers on Wall for Test 3451-35.

14,524 kg/91.6 kph/15.8 deg
(32,020 lb/56.9 mph/15.8 deg)

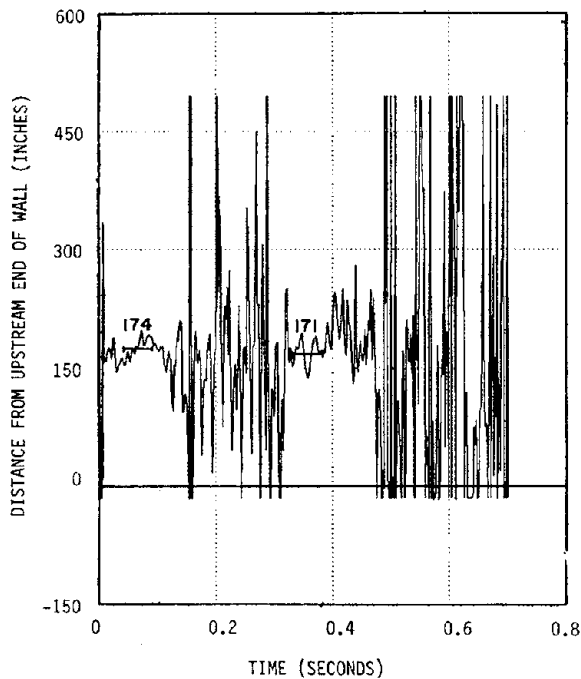


Figure 92. Horizontal Position of Total Lateral Force on the Wall for Test 3451-35.

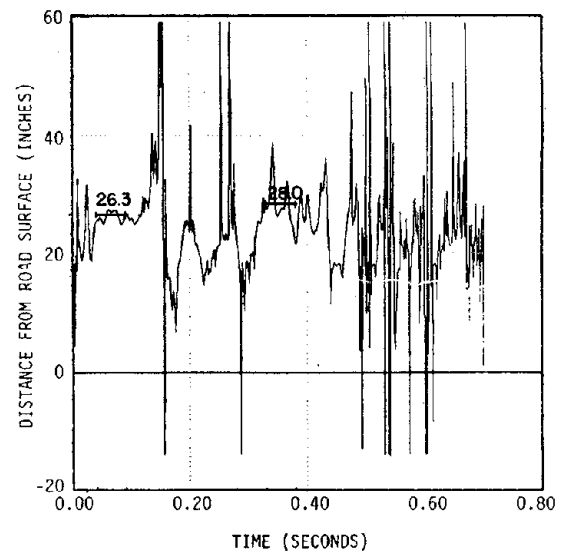


Figure 93. Vertical Position of Total Lateral Force on the Wall for Test 3451-35.

ient strength to redirect school busses but sometimes do not have adequate height to provide needed stability for the bus.

The intercity bus imposed significantly higher forces than either the automobile or the school bus and it has been observed in full-scale tests on typical bridge railings that strength of the railing is generally inadequate for the intercity bus.

Figures 94 and 95 show relationships between lateral forces measured by wall instrumentation and lateral forces measured by vehicle instrumentation.

The manner in which forces were distributed along the length of the railing and in the vertical direction was then addressed. It was assumed that the force was distributed in the form of a half sine wave in both the longitudinal and vertical directions. This assumption yielded the equation:

$$R = \int_0^D \int_0^L q_{\max} \sin \frac{\pi x}{L} \sin \frac{\pi y}{D} dy dx \quad \text{Eqn 1}$$

where q_{\max} = Maximum bearing intensity, k/ft².

R = Resultant force, k

D = Vertical extent of the loaded area, ft

L = Longitudinal extent of the loaded area, ft

The coordinates x and y, of the rectangular contact area are as shown in Figure 96. The length of the contact area was measured from the plan view movie frame that fell nearest the center of 0.050 sec time interval (for both the initial and final impact). The depth dimension was deduced by subtracting the 0.08 m (3 in.) sill height from the height to the resultant in Table 9 to find D/2 or, when the resultant lay above 0.57 m (39/2 + 3 = 22.5 in.), (the midheight of the wall segments) by subtracting the resultant height from 1.07 m (42 in.) to find D/2. When integrated and inverted to solve for the maximum intensity in terms of the measured resultant one finds that:

$$q_{\max} = R \pi^2 / 4DL \quad \text{Eqn 2}$$

A typical distribution, (for the initial impact of the 2,150 kg (4,740 lb) vehicle at 96.2 kph (59.8 mph) and 24 deg) is shown in Figure 97.

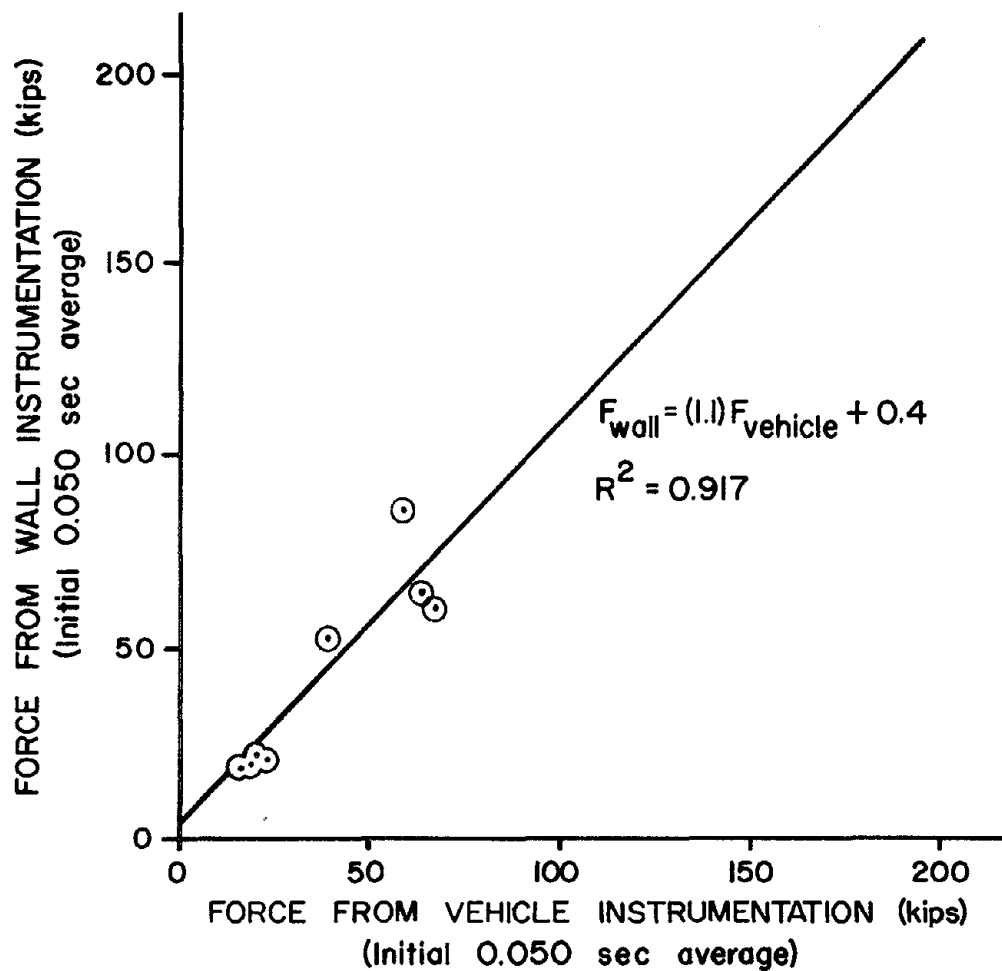


Figure 94. Comparison of Forces from Wall Instrumentation with forces from vehicle instrumentation for initial impact using vehicle test inertia mass.

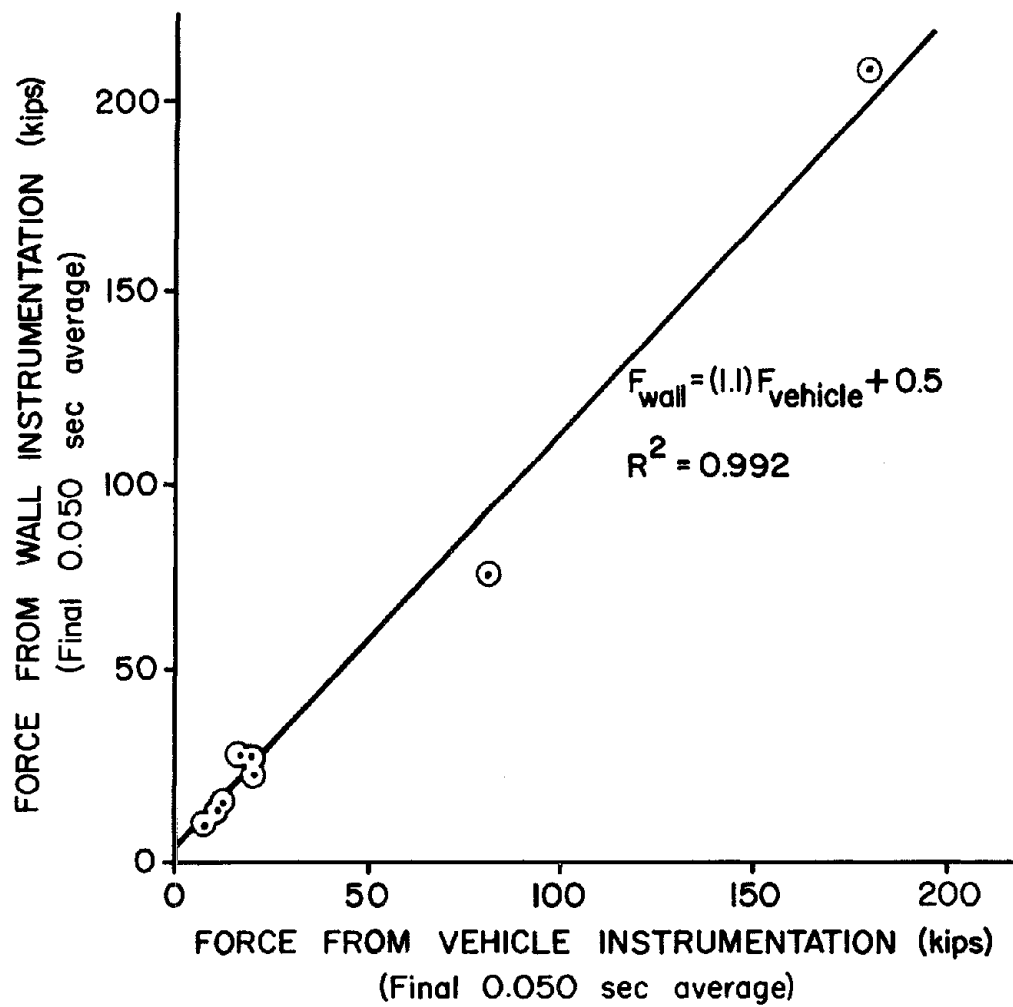
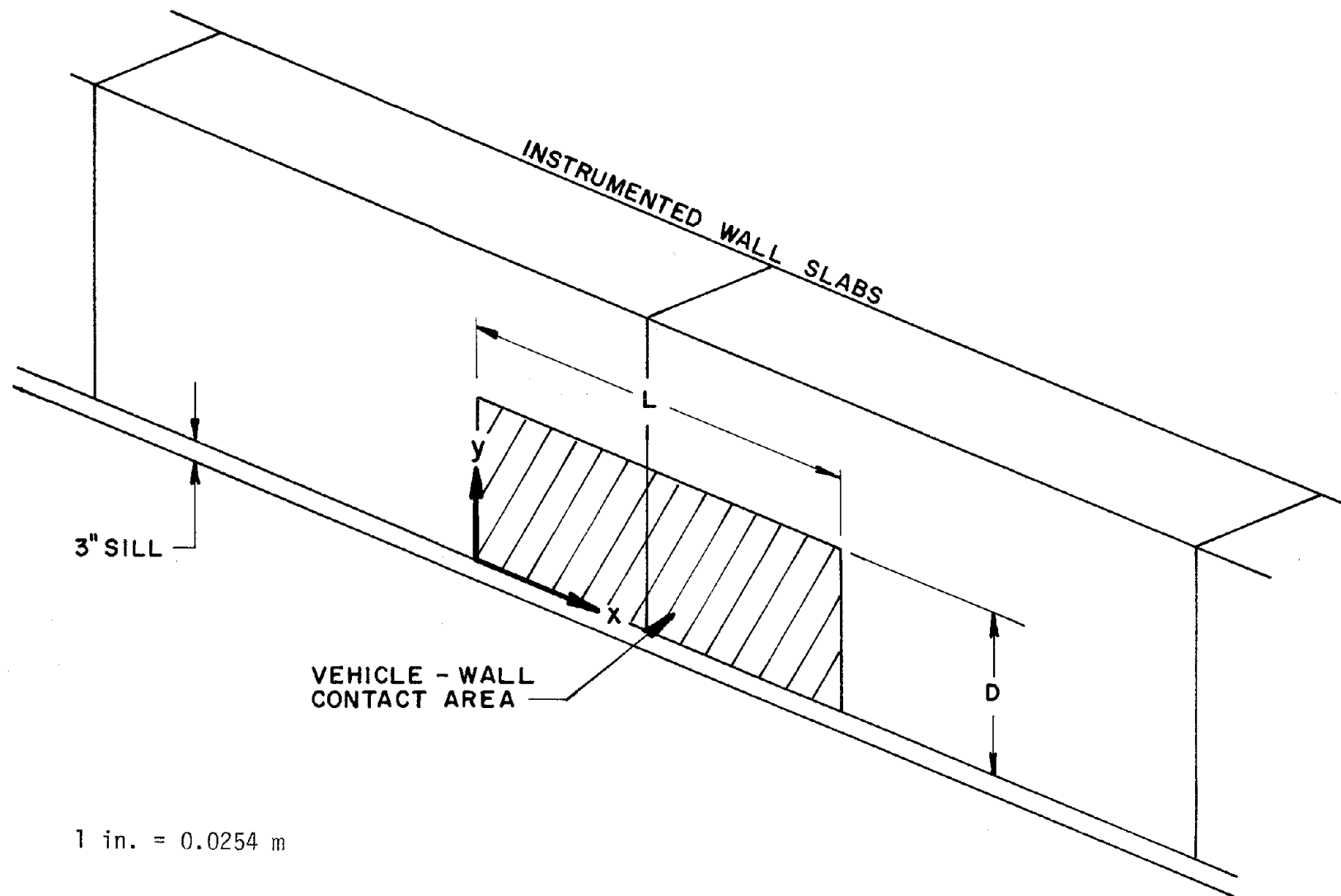
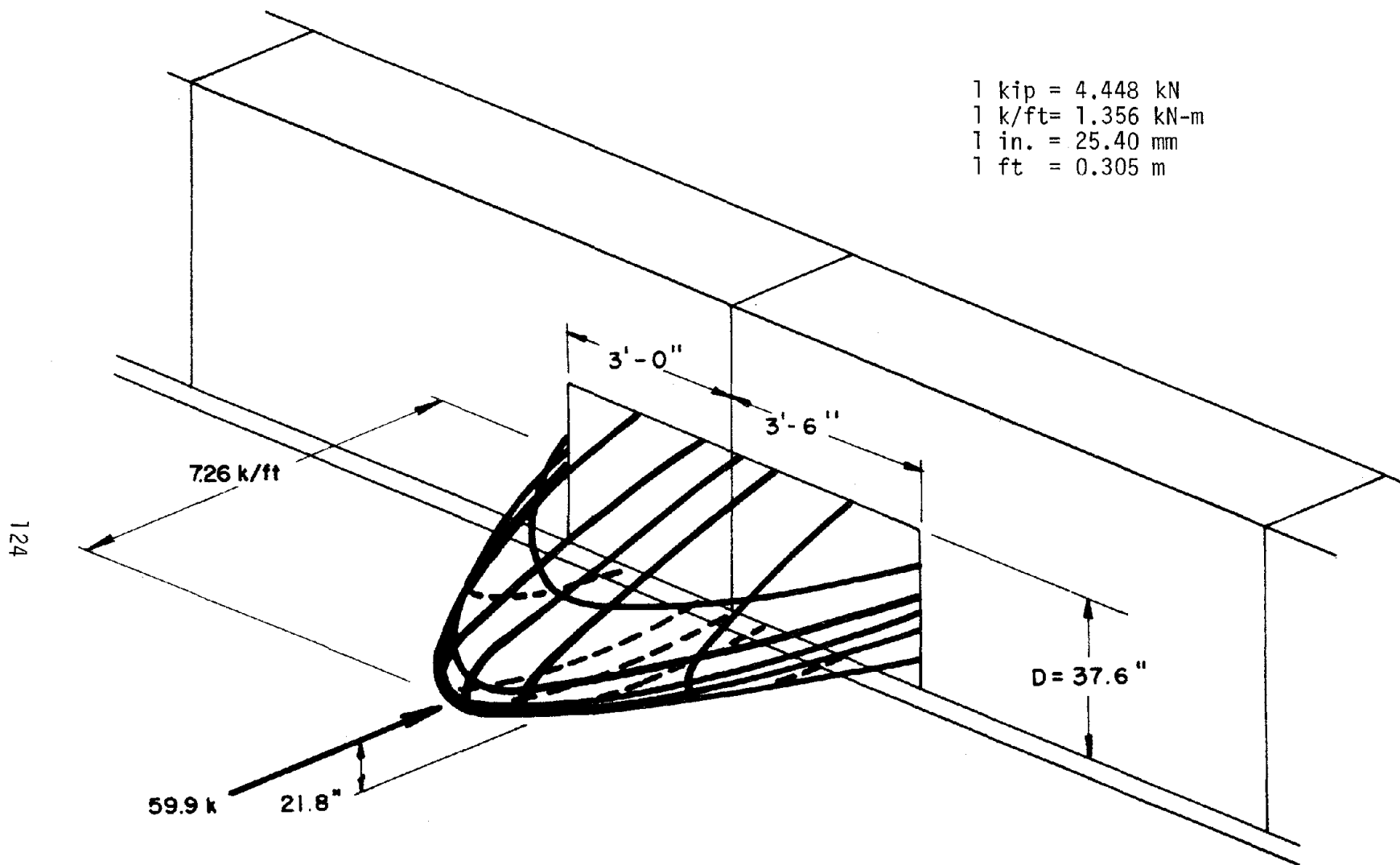


Figure 95. Comparison of Forces from Wall Instrumentation with forces from vehicle instrumentation for final impact using vehicle test inertia mass.



1 in. = 0.0254 m

Figure 96 . Sketch Showing Rectangular Contact Area on Instrumented Wall.



To calculate the magnitude of the maximum vehicle to wall pressure:

$$q_{\max} = (59.9 \text{ k}) \frac{\pi^2}{4DL} = 7.26 \text{ k/ft}^2$$

Figure 97 . Sketch Indicating the Assumed Distribution of Contact Pressures. Illustrative Values are for Test 3451-36.

Computed maximum contact pressures and the associated lengths and heights are summarized in Table 10.

The maximum values contact pressures given in Table 10 may be converted to maximum values of distributed load in kips per ft along the length of the railing by multiplying the pressures by $2D/\pi$:

$$W_{\max} = q_{\max} \frac{2D}{\pi} \quad \text{Eqn 3}$$

These computations were performed and the results are presented in Table 11. It is observed that these measured dynamic impact forces are significantly higher than static loads which are used in designing railing systems for automobiles. This is not unexpected because a railing system designed for a static load of 45 kN (10 kips) using allowable stress design procedures would be expected to exhibit an ultimate strength (or a reserve capacity) well above 45 kN (10 kips.). However, the amount of reserve capacity will differ for various materials and various design details such as connections between members, and is not really predicted when allowable stress design procedures are used. Ultimate strength design procedures will allow more accurate prediction of the actual strength of a structure.

It is recommended that new bridge railing designs be based on ultimate strength design procedures using yield strength of the material with a factor of safety equal to 1.0 in conjunction with the impact loads presented in Table 12. Such a procedure will produce yielding but not fracture when a collision occurs, assuming the material and structural elements have adequate ductility and ultimate strength much greater than yield strength. Failure mechanisms that are appropriate for the different sizes of vehicles and types of railing may be determined from full-scale crash tests reported herein and in other reports (6, 7) or from full-scale crash tests on a proposed railing. Values given in Table 12 are for basically "rigid" railing systems and they are not recommended for application to flexible railing systems. A railing system that deflects significantly would be subjected to lower loads because of the energy absorbed during deflection. Determination of design load values for flexible railing systems was beyond the scope of this study.

An example of application of the recommended design guidelines has been performed by Hirsch in another study (7) and is summarized in Appendix D. Additional work on the subject is presented in references 6 and 7.

Table 10. Maximum Contact Pressures Deduced from the Instrumented Wall Tests.

Test Condition	Initial Impact			Final Impact		
	Max. Contact Pressure (k/ft ²)	Length (ft)	Height (ft)	Max. Contact Pressure (k/ft ²)	Length (ft)	Height (ft)
1,970 lb/59.0 mph/15.5 deg	3.89	5.00	2.33	1.11	7.58	2.58
2,090 lb/58.5 mph/21.0 deg	3.25	6.00	2.67	1.35	8.00	3.00
2,800 lb/58.3 mph/14.8 deg	3.85	5.00	2.50	1.82	10.83	2.08
2,830 lb/56.0 mph/20.0 deg	3.65	4.00	2.92	1.52	10.17	3.00
4,680 lb/54.6 mph/16.5 deg	5.73	7.33	3.08	2.01	10.67	3.25
4,740 lb/59.8 mph/24.0 deg	7.26	6.50	3.13	1.48	14.50	3.25
20,030 lb/57.6 mph/16.5 deg	5.88	12.33	2.17	6.78	17.00	1.58
32,020 lb/56.9 mph/15.8 deg	12.90	6.25	2.58	15.40	15.00	2.25

1 lb = 0.454 kg

1 mph = 1.609 kph

1 ft = 0.305 m

1 k/ft = 1.356 kN-m

Table 11. Maximum Force Per Unit Length of Railing from the Instrumented Wall Tests.

Test Condition	ISP* (ft-k)	Initial Impact		Final Impact	
		Max. Force Per Unit Length (k/ft)	Length (ft)	Max. Force Per Unit Length (k/ft)	Length (ft)
1,970 lb/59.0 mph/15.5 deg	14.4	5.76	5.00	1.82	7.58
2,090 lb/58.5 mph/21.0 deg	26.0	5.52	6.00	2.58	8.00
2,800 lb/58.3 mph/14.8 deg	18.9	5.81	5.00	2.01	10.83
2,830 lb/56.0 mph/20.0 deg	26.5	7.61	4.00	3.48	10.17
4,680 lb/54.6 mph/16.5 deg	27.3	11.24	7.33	4.16	10.67
4,700 lb/58.9 mph/23.8 deg	87.6	14.49	6.50	3.06	14.50
20,030 lb/57.6 mph/16.5 deg	96.7	8.12	12.33	6.82	17.00
32,020 lb/56.9 mph/15.8 deg	166.5	21.20	6.25	22.10	15.00

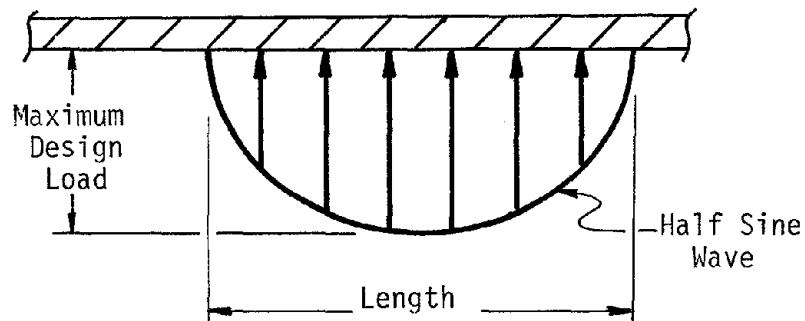
$$*ISP \text{ (Impact Severity Potential)} = 1/2 mV^2 \sin^2 \theta$$

1 lb = 0.454 kg
 1 mph = 1.609 kph
 1 ft = 0.305 m
 1 k/ft = 1.356 kN-m

Table 12. Recommended Ultimate Design Loads for Bridge Railings That Do Not Deflect Significantly.

Design Test Condition	Max. Design Load (k/ft)	Length of Load (ft)	Effective Height (in.)
4,500 lb/60 mph/15 deg	11.2	7.3	24
4,500 lb/60 mph/25 deg	14.5	6.5	24
20,000 lb/60 mph/15 deg	8.1	12.3	34
32,000 lb/60 mph/15 deg	22.1	15.0	30

1 lb = 0.454 kg
 1 mph = 1.609 kph
 1 k/ft = 1.356 kN-m
 1 ft = 0.305 m
 1 in. = 0.025 m



IV. PERFORMANCE STANDARDS

Design guidelines for bridge railing systems have been developed and are presented in Chapter III of this report. These guidelines are limited in scope as to the spectrum of vehicles considered and address only strength and geometric features of the railing system above the deck. The guidelines represent the best effort based on current technology and information gained from the research program reported herein. In some aspects, the guidelines are based on limited data and do not provide validated assurance that an adequate railing system will be achieved. Because of these limitations and the performance record of in-service railing systems it is recommended that all bridge railing systems be required to meet certain performance standards based on full-scale crash tests.

Performance standards for bridge railing systems were developed (See Appendix E) and are presented here. These standards are based on results obtained from tests on the flat-faced instrumented wall and a relationship between occupant injury and vehicle damage. These standards are recommended as replacements for the corresponding items in NCHRP Report 230 (9) and would be used along with the test procedures now contained in NCHRP Report 230 (9) in evaluating the performance of bridge railing systems for the spectrum of vehicles addressed herein.

A relationship between highest 0.050 sec avg longitudinal vehicle acceleration and impact angle from tests on the instrumented wall is presented in Figure 98. This relationship was used to arrive at recommended allowable longitudinal vehicle acceleration values. The recommended values are nominally 125 percent of the highest test values. It is reasoned that these limits on vehicle longitudinal acceleration will provide appropriate quantitative discrimination of unacceptable snagging and that railings meeting these limits along with other appropriate and necessary requirements will provide smooth redirection of automobiles.

The real purpose of any proposed performance standard, and in fact, the basic need for installing a railing system on a bridge structure is to contain vehicles and to do so with the lowest possible severity or probability of injury to the occupants. The need to minimize severity of

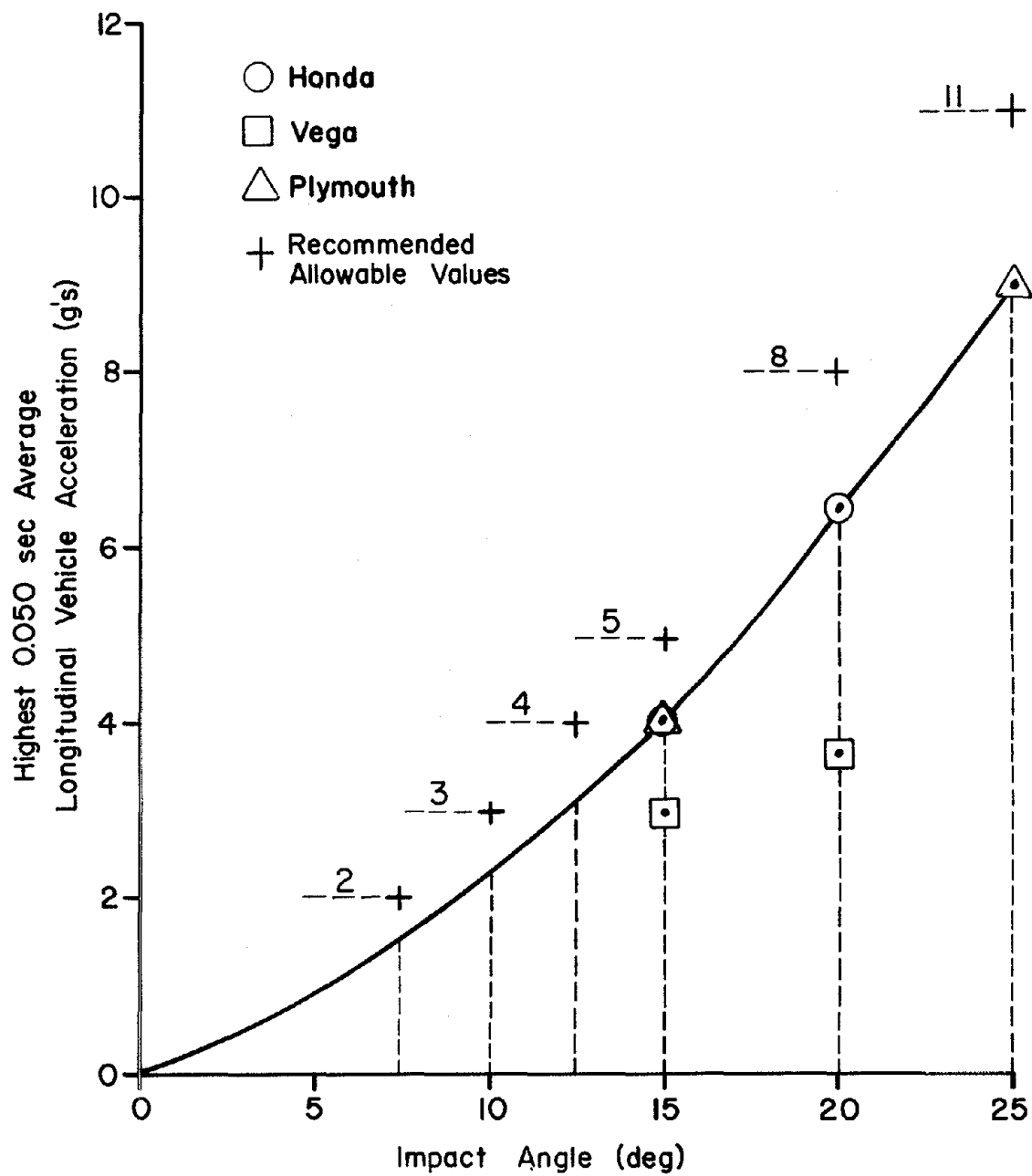


Figure 98. Relationship of Recommended Allowable Vehicle Longitudinal Accelerations to Those Determined from Tests on Instrumented Wall.

injury has prompted the use of instrumented dummies and the development of acceptance criteria based on the responses of dummies during a crash test. The success of this approach has not been satisfactory for many reasons, one of them being the extremely poor relationship between dummy responses and injuries sustained by live occupants.

An alternate approach that has been considered is to relate probability or severity of injury to some other parameter that has been quantified and is common to both accidents and crash tests. The most promising parameter of this type is damage to the vehicle.

Michalski (28), in 1968, reported the relationship between proportion of cars in which injuries occurred and vehicle front-end damage shown in Figure 99. Although more current data reflecting the improvements in vehicle interior design would be preferred, the Michalski relationship is still the best available and has the possible advantage that it may be conservative. Since it is based on vehicles with less forgiving interiors, the probability of injuries should be even lower in contemporary vehicles, if the vehicle size effect is ignored. However, the effect of reduction in vehicle size would tend to counteract improvements due to interior design. The net result is that the Michalski relationship may be as good today as it was in 1968.

This relationship together with data from recent crash tests can be used to develop standards for determining acceptability of performance of railing systems. A plot of automobile damage rating vs. automobile resultant acceleration is shown in Figure 100. Equations in this figure and in Figure 99 can be combined to obtain the equation which was used to construct the line in Figure 101.

It should be noted that this analysis is based on somewhat dated and limited accident data. Because of these limitations the resulting relationship could probably be improved through analyses of more recent and extensive accident data. Such data are recorded and are available from selected States. Efforts are now underway to begin this analysis on Texas data.

There are several candidate techniques for establishing limits on a crash test result such as vehicle resultant acceleration. One technique would be to establish an acceptable probability of injury level for bridge

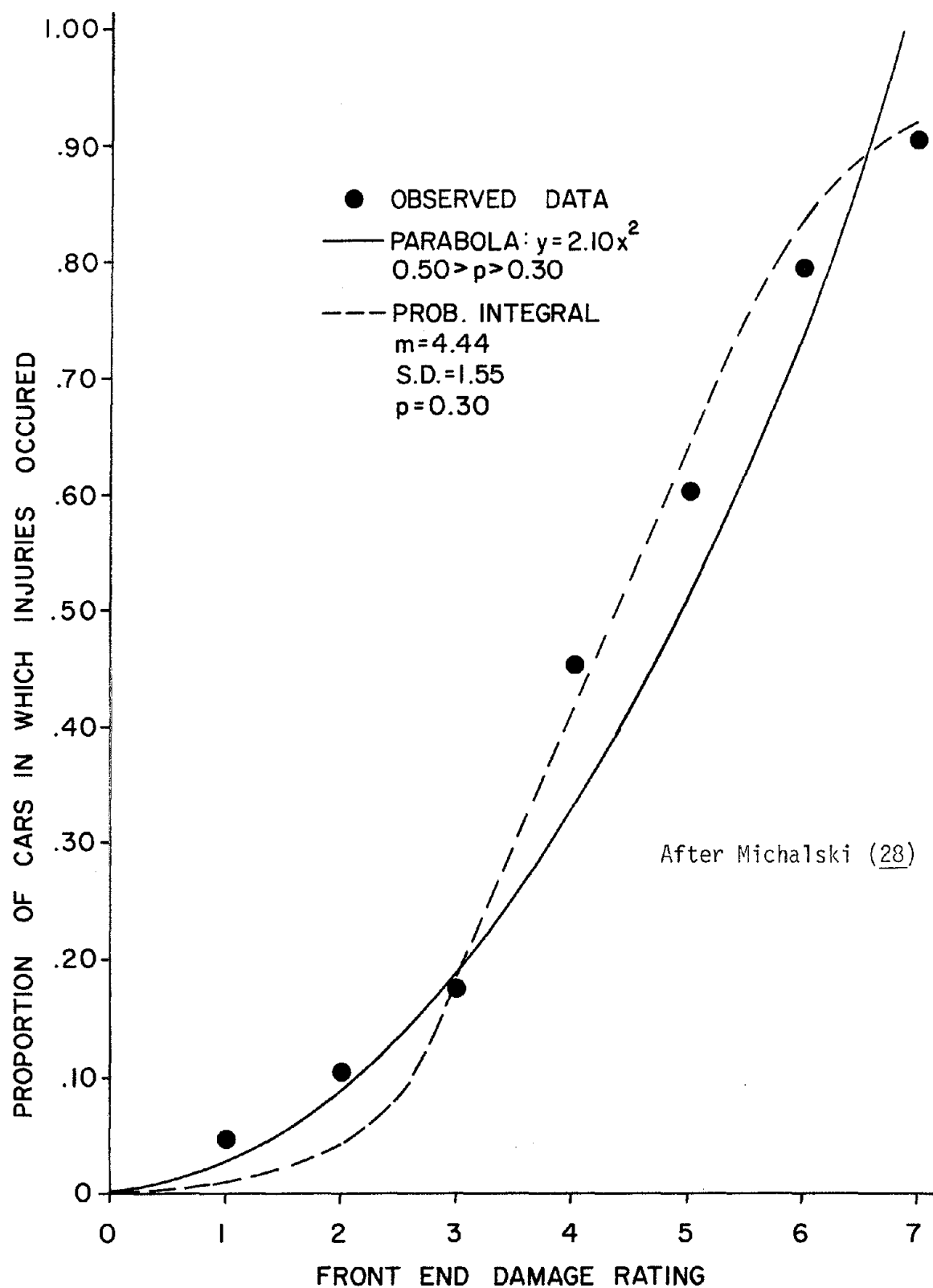


Figure 99. Occurrence of Personal Injuries in Relation to Vehicle Front-End Damage Rating.

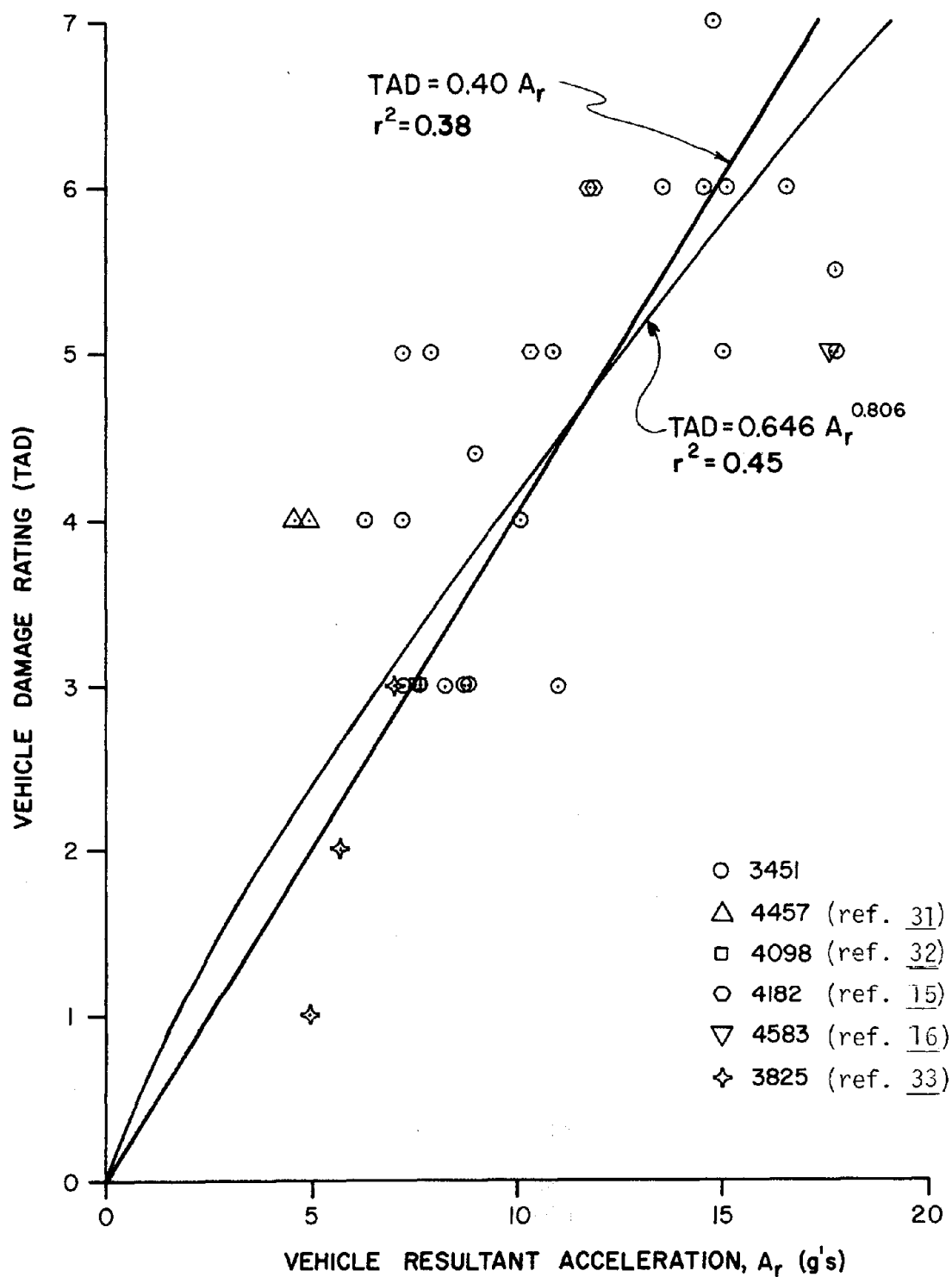


Figure 100. Relationship Between Vehicle Damage Rating and Vehicle Resultant Acceleration.

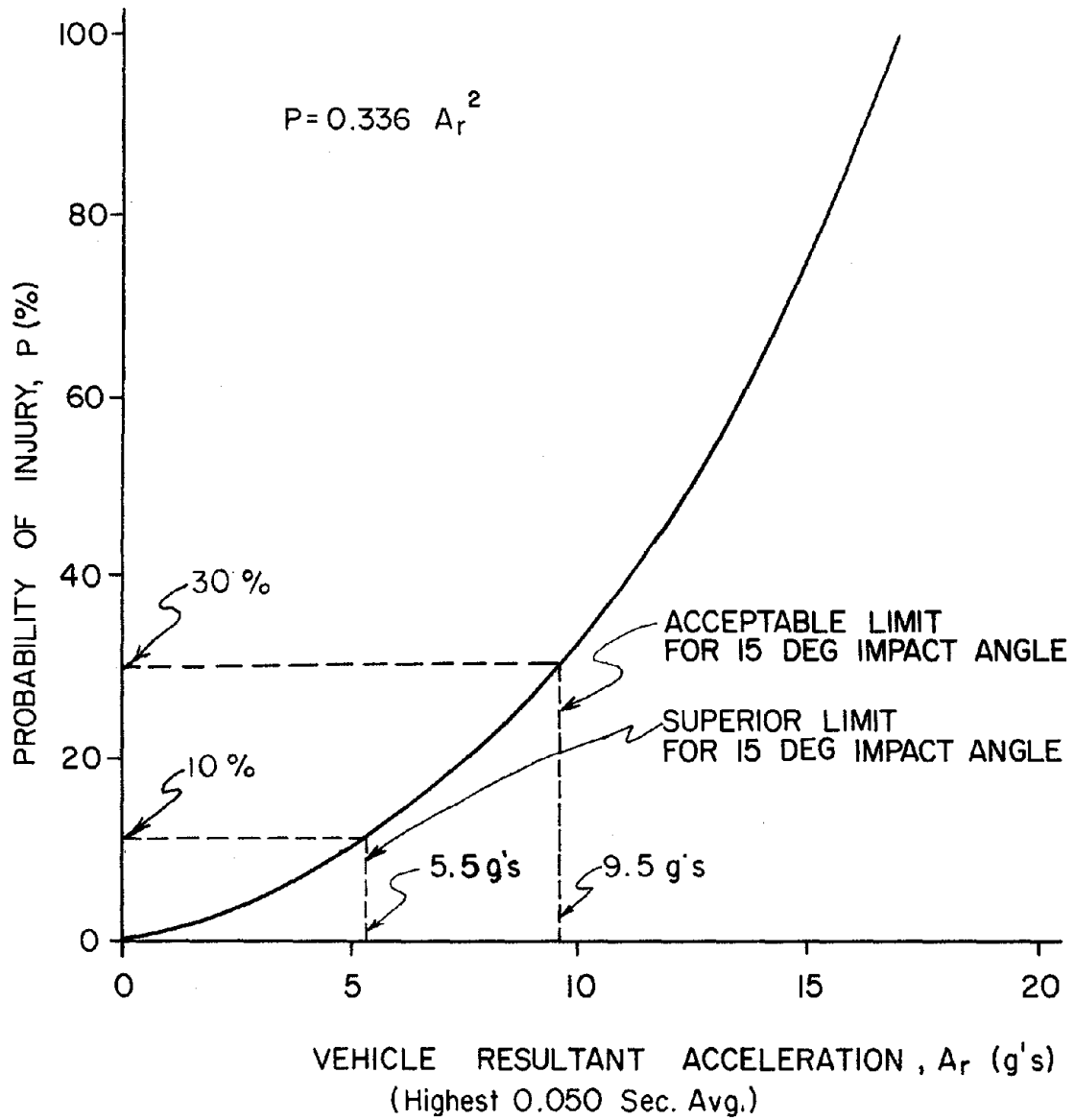


Figure 101. Relationship Between Probability of Injury and Automobile Resultant Acceleration.

railing impacts and, using the relationship in Figure 101, establish an acceptable vehicle resultant acceleration. Another technique would be to use crash test results, from railing designs having certain characteristics, to directly establish acceptable vehicle resultant accelerations.

The test conditions to which these limits are to be applied must also be determined.

If it were established that a 30 percent probability of injury were acceptable, Figure 101 would indicate that a vehicle resultant acceleration of up to 9.5 g's would be acceptable. A limit at or near this value (say 10 g's) might be appropriate for determining acceptability of rigid, nondeflecting railing designs.

One might further extend these standards to include a limit for superior railing system designs of, say, a 10 percent probability of injury which would correspond to a vehicle resultant acceleration of 6 g's.

Recommended evaluation criteria for determining acceptability of performance of bridge railings are presented in Table 13.

These criteria were applied to the test results reported herein and the acceptability of performance is shown in Table 14.

The question that now needs to be considered is whether the evaluation criteria result in the "correct answer". This question can not be answered with any degree of confidence unless one has in-service accident histories available for railing systems meeting and failing the criteria and can study possible relationships. The best that one can do at this time is to qualitatively examine the acceptability/unacceptability decision of the performance criteria. The Colorado railing is rejected in all but one test and this appears to be the appropriate decision. The Texas T101 railing passes in all tests except one with an intercity bus which rolled onto its side during the collision. The New Hampshire railing failed to meet the requirements in all tests except one with a 2,043 kg (4,500-lb) vehicle at a 15-deg impact angle. In this test, vehicle accelerations were marginal. Only one test, a school bus test, was performed on the North Carolina railing and that vehicle rolled. This railing would be expected to demonstrate acceptable performance in tests with automobiles. The Indiana 5A railing passed the requirements in two tests with automobiles.

Table 13. Recommended Evaluation Criteria
for Determining Acceptability
of Performance of Bridge Railings.

- (1) Test article shall smoothly redirect the vehicle; the vehicle shall not penetrate or go over the installation although controlled lateral deflection of the test article is acceptable. Precluding significant maintenance problems, controlled lateral deflection is encouraged.
- (2) The highest 0.050 second average longitudinal and resultant acceleration imposed on the vehicle shall not exceed the "acceptable" levels given below. Resultant accelerations below or equal to the "superior" levels indicate lower probabilities of occupant injury.

Acceptable and Superior Levels of Performance			
Automobile Test Condition	Maximum Allowable Highest 0.050 Sec Avg. Longitudinal Accel.	Maximum Allowable Highest 0.050 Sec Avg Resultant Accel.	
Speed/Angle mph/degrees	Acceptable g's	Acceptable g's	Superior g's
60/15	5	10	6
60/20	8	N.A.	8
60/25	11	N.A.	10

- (3) Detached elements, fragments or other debris from the test article shall not penetrate or show potential for penetrating the passenger compartment or present undue hazard to other traffic.
- (4) The vehicle shall remain upright during and after collision although moderate roll, pitch and yaw displacements are acceptable. Integrity of the passenger compartment must be maintained with essentially no deformation or intrusion having potential for resulting in injury to occupants.

Note: An objective definition of "smoothly redirect" is given by the acceptable longitudinal acceleration levels of Article 2. Compliance with these longitudinal acceleration levels eliminates the need for subjective interpretation of the "smoothly redirect" criterion in Article 1, in questionable cases.

Table 14. Summary of Acceptability of Performance of Bridge Railings by Recommended Performance Standards.

TEST NUMBER	TEST CONDITIONS	RAILING DESIGN	EVALUATION CRITERIA						EVALUATION OF PERFORMANCE
			(1)	(2)		(3)	(4)		
				Long.	Result.				
3451-01	2,770 lb/56.0 mph/15.1°	Colorado Type 5	Fail	Fail	Acc.	Pass	Pass	Fail	
3451-02	4,700 lb/62.8 mph/15.0°	Colorado Type 5	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-03	4,640 lb/61.4 mph/24.5°	Colorado Type 5	Fail	Fail	N/App	Pass	Pass	Fail	
3451-04	19,760 lb/59.4 mph/14.3°	Colorado Type 5	Pass	N/App	N/App	Pass	Fail	Fail	
3451-05	2,780 lb/57.3 mph/15.0°	Texas T101	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-06	4,660 lb/60.2 mph/15.0°	Texas T101	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-07	4,630 lb/59.8 mph/25.8°	Texas T101	Pass	Pass	Sup.	Pass	Pass	Sup.	
3451-08	6,900 lb/53.4 mph/15.0°	Texas T101	Pass	N/App	N/App	Pass	Pass	Pass	
3451-09	19,940 lb/55.3 mph/15.2°	Texas T101	Pass	N/App	N/App	Pass	Pass	Pass	
3451-10	20,010 lb/52.0 mph/13.2°	Texas T101	Pass	N/App	N/App	Pass	Pass	Pass	
3451-11	31,880 lb/58.4 mph/16.0°	Texas T101	Pass	N/App	N/App	Pass	Fail	Fail	
3451-12	1,950 lb/60.9 mph/15.0°	New Hampshire	Fail	Fail	Fail	Pass	Fail	Fail	
3451-13	2,780 lb/58.4 mph/15.0°	New Hampshire	Fail	Fail	Fail	Pass	Pass	Fail	
3551-14	2,780 lb/59.1 mph/20.5°	New Hampshire	Fail	Fail	N/App	Pass	Pass	Fail	
3451-15	4,670 lb/59.2 mph/15.0°	New Hampshire	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-23	19,920 lb/57.3 mph/14.8°	North Carolina	Pass	N/App	N/App	Pass	Fail	Fail	
3451-24	1,950 lb/57.5 mph/12.5°	Indiana 5A	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-25	2,780 lb/53.6 mph/19.5°	Indiana 5A	Pass	Pass	N/App	Pass	Pass	Pass	
3451-26	4,670 lb/61.6 mph/25.8°	Indiana 5A	Fail	Fail	N/App	Pass	Pass	Fail	
3451-27	2,150 lb/54.8 mph/20.0°	Indiana 5A	Fail	Fail	N/App	Pass	Pass	Fail	
3451-28	2,050 lb/55.4 mph/19.0°	Mod. Indiana 5A	Fail	Fail	N/App	Pass	Pass	Fail	
3451-29	1,970 lb/59.0 mph/15.5°	Instrumented Wall	Pass	Pass	Fail	Pass	Pass	Fail	
3451-30	2,800 lb/58.3 mph/14.8°	Instrumented Wall	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-31	2,830 lb/56.0 mph/20.0°	Instrumented Wall	Pass	Pass	N/App	Pass	Pass	Pass	
3451-32	4,680 lb/54.6 mph/16.5°	Instrumented Wall	Pass	Pass	Acc.	Pass	Pass	Acc.	
3451-33	4,700 lb/58.9 mph/23.8°	Instrumented Wall	No Data	No Data	No Data	Pass	Pass	No Data	
3451-34	20,030 lb/57.6 mph/16.5°	Instrumented Wall	Pass	N/App	N/App	Pass	Pass	Pass	
3451-35	32,020 lb/56.9 mph/15.8°	Instrumented Wall	Pass	N/App	N/App	Pass	Pass	Pass	
3451-36	4,740 lb/59.8 mph/24.0°	Instrumented Wall	Pass	Pass	N/App	Pass	Pass	Pass	
3451-37	2,090 lb/58.5 mph/21.0°	Instrumented Wall	Pass	Pass	N/App	Pass	Pass	Pass	

1 lb = 0.454 kg

1 mph = 1.609 kph

Evaluation of Recommended Performance Standards

One of the tasks of this study was to evaluate the recommended performance standards by applying them to full-scale crash tests on selected railing systems and determining the acceptability of performance of these systems.

Literature and research reports were reviewed in search of railing designs which had been subjected to full-scale tests and which spanned a range of performance level. The search resulted in a "shopping list" from which representatives of the Federal Highway Administration chose four. These included the Safety Shape (4), Tubular Thrie-Beam on Tubes (1), Collapsing Ring (21), and Low Service Level Thrie Beam (20).

Safety Shape (New Jersey Profile)

This design is very popular and in wide-spread use, both as a bridge railing and as a median barrier (4). A cross-section of a concrete safety shape bridge railing is shown in Figure 102 and a summary of acceptability of this barrier design, based on the recommended performance standards, is listed in Table 15. The concrete safety shape failed to meet the recommended performance standards in a 2,041 kg/97 kph/25 deg (4,500 lb/60 mph/25 deg) test because of lack of smoothness of redirection and excessive roll displacement (60 deg away from the barrier) of the vehicle. This design meets requirements for less severe test conditions with automobiles and performs adequately in low angle tests with an intercity bus at lower impact speeds.

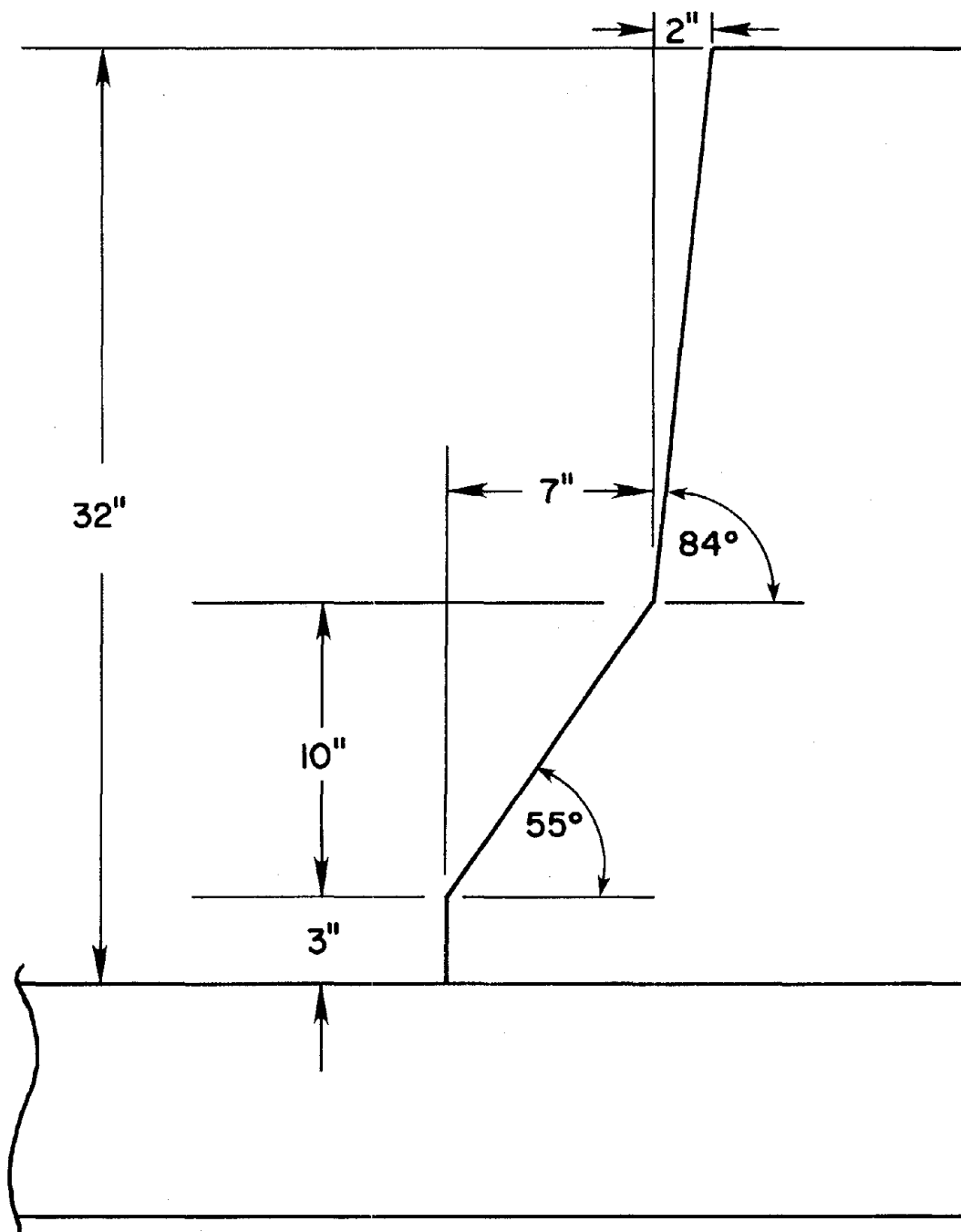
Tubular Thrie-Beam on Tubes

This is a retrofit railing with several design versions developed by Southwest Research Institute(1, 34). It incorporates thin-walled tubes for energy absorbers (Figure 103).

A summary of acceptability of this railing, based on the recommended performance standards, is listed in Table 16. Designs R(II)-1 and R(IIIN)-1 meet requirements for tests with automobiles at test conditions as severe as 2,041 kg/97 kph/25 deg (4,500 lb/60 mph/25 deg).

Collapsing Ring

An extensive series of full-scale crash tests has been conducted on this railing design (21). As shown in Figure 104, the design is relatively intricate. The initial cost of this system is relatively high and it has had very limited implementation to date. It would be best suited for



1 in. = 0.025 m
1 in. = 25.40 mm

Figure 102. Safety Shape
(New Jersey Profile).

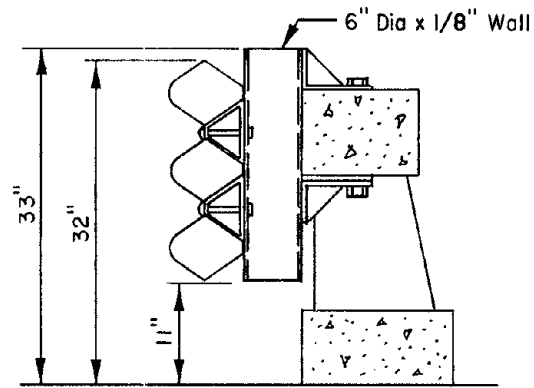
Table 15. Summary of Acceptability of Safety Shape
by Recommended Performance Standards.

Test Designation	Test Conditions	Evaluation Criteria*				(4)	Evaluation of Performance
		(1)	(2)		(3)		
			Long.	Result.			
CMB 4	4,370/55.9/15.9	Pass	Pass	Sup.	Pass	Pass	Sup.
CMB 9	2,250/58.9/15.5	Pass	Pass	Sup.	Pass	Pass	Sup.
CMB 15	4,500/60.0/25.0	Pass	Pass	N/App	Pass	Fail	Fail
CMB 23	40,000/52.9/16.0	Pass	N/App	N/App	Pass	Pass	Pass
3115-1	19,990/60.9/16.0	Pass	N/App	N/App	Pass	Fail	Fail
3115-2	1,970/60.4/15.0	Pass	Pass	Acc.	Pass	Pass	Acc.
3115-3	1,968/61.3/20.0	Pass	Pass	Sup.	Pass	Pass	Sup.
8307-1	40,020/54.0/16.2	Pass	N/App	N/App	Pass	Pass	Pass
8307-3	40,030/54.0/14.0	Pass	N/App	N/App	Pass	Pass	Pass

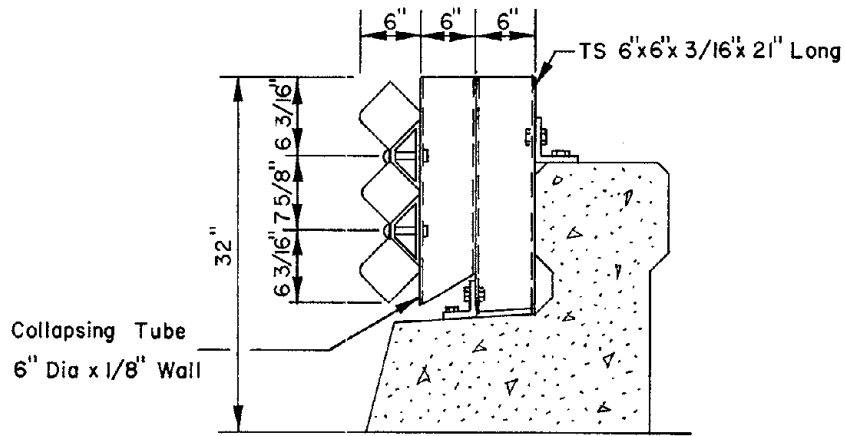
*Refers to Table 13

1 lb = 0.454 kg

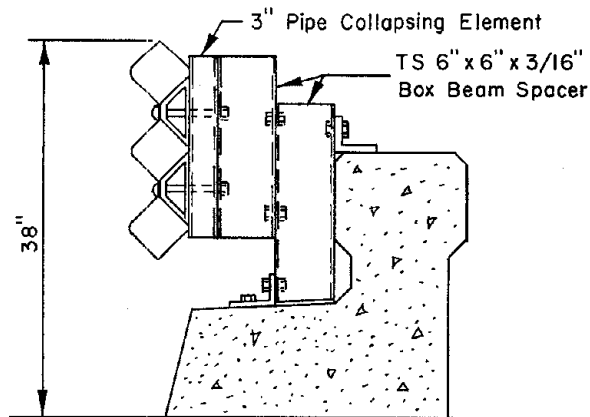
1 mph = 1.609 kph



(a) Design R(II)-1



(b) Design R(IIIN)-1



(c) Modified Retrofit Design

Figure 103. Tubular Thrie-Beam on Tubes.

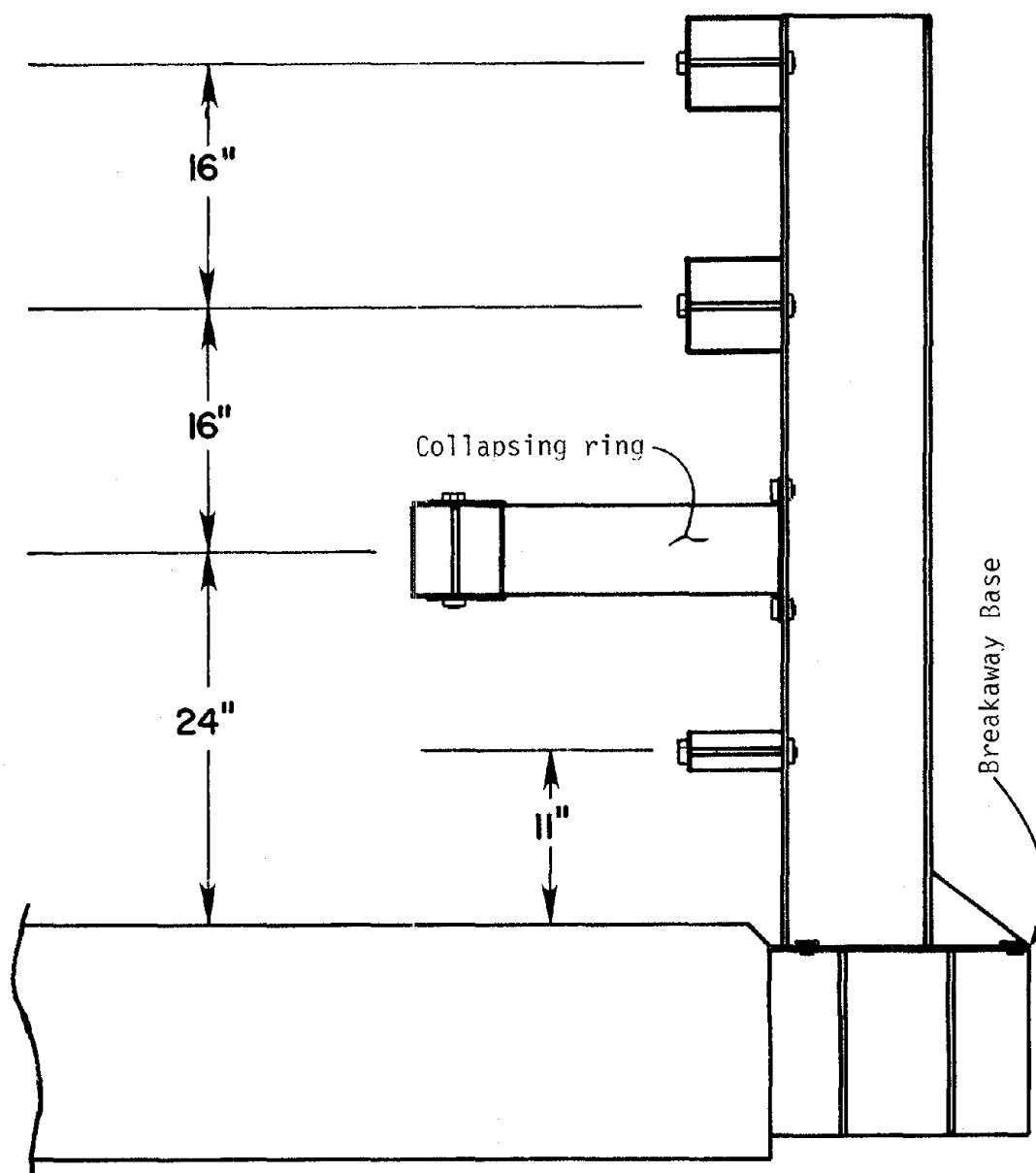
Table 16. Summary of Acceptability of Tubular Thrie-Beam
on Tubes by Recommended Performance Standards.

Test Designation	Railing Design	Test Conditions	Evaluation Criteria*					Evaluation of Performance
			(1)	(2)		(3)	(4)	
				Long.	Result.			
RF 5	R(II)-1	2,250/58.0/17.1	Pass	Pass	Acc.	Pass	Pass	Acc.
RF 6	R(II)-1	4,500/60.6/25.0	Pass	Pass	Acc.	Pass	Pass	Acc.
RF 1	R(IIIN)-1	2,140/63.6/16.8	Pass	Pass	Acc.	Pass	Pass	Acc.
RF 2	R(IIIN)-1	4,300/66.6/23.9	Pass	Pass	Acc.	Pass	Pass	Acc.
RF 28	Modified Retrofit	40,000/56.3/14.5	Pass	N/App	N/App	Pass	Pass	Pass
RF 29	Modified Retrofit	1,840/58.1/18.8	Pass	Pass	Acc.	Pass	Pass	Acc.

*Refers to Table 13

1 lb = 0.454 kg

1 mph = 1.609 kph



1 in. = 0.025 m
 1 in. = 25.40 mm

Figure 104. Collapsing Ring Bridgerail System.

installation sites where a rather high level of service is needed and can be afforded.

A summary of acceptability of this railing design, based on the recommended performance standards, is listed in Table 17. All, except one, reported tests on the collapsing ring meet the recommended performance criteria. In the one test that fails, a premature structural failure occurred in a transverse weld. Transverse welds are prohibited in many state standards. Tests with tractor/trailer units are not addressed by the recommended performance criteria.

Low Service Level Thrie-Beam

This design was also developed by Southwest Research Institute (20). It consists of a thrie-beam mounted on either wood or steel posts. The posts are side-mounted on the deck (Figure 105). The design is for service level 1 which requires a structural adequacy test with a 2,041 kg (4,500 lb) automobile at 97 kph (60 mph) and 15 deg (20).

A summary of acceptability of this railing design, based on the recommended performance standards, is listed in Table 18. All reported test results on the low service level thrie-beam railing meet the recommended performance standards and longitudinal accelerations imposed on the vehicle meet the superior levels.

Table 17. Summary of Acceptability of Collapsing Ring
by Recommended Performance Standards.

Test Designation	Test Conditions	Evaluation Criteria*					Evaluation of Performance
		(1)	(2)		(3)	(4)	
			Long.	Result.			
BR 3	3,960/60.0/24.7	Pass	Pass	Acc.	Pass	Pass	Acc.
BR 4	4,097/60.6/25.9	Fail	Pass	Sup.	Pass	Fail	Fail
BR 5	3,910/56.1/23.9	Pass	Pass	Sup.	Pass	Pass	Sup.
BR 6	2,090/55.7/23.5	Pass	Pass	Acc.	Pass	Pass	Acc.
BR 7	4,230/56.7/29.1	Pass	Pass	Sup.	Pass	Pass	Sup.
BR 8	19,000/60.9/13.9	Pass	N/App	N/App	Pass	Pass	Pass
BR 9	40,000/54.3/19.1	Pass	N/App	N/App	Pass	Pass	Pass
BR 10	40,000/55.1/19.0	N/App	--	--	N/App	N/App	--
BR 11	40,000/54.9/15.1	Pass	N/App	N/App	Pass	Pass	Pass
BR 13	4,400/62.0/22.7	Pass	Pass	Sup.	Pass	Pass	Sup.
BR 14	40,000/57.0/15.6	N/App	--	--	N/App	N/App	--

*Refers to Table 13

1 lb = 0.454 kg

1 mph = 1.609 kph

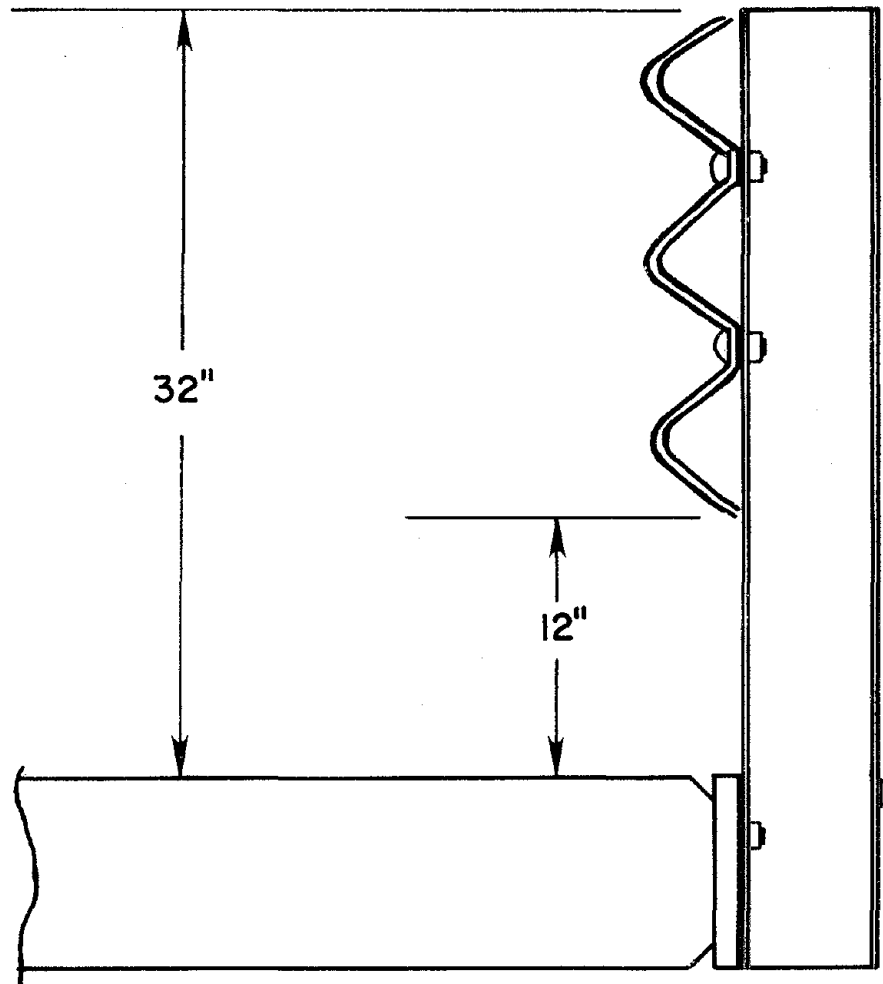


Figure 105. Low Service Level Thrie-Beam.

Table 18. Summary of Acceptability of Low Service Level
Railing by Recommended Performance Standards.

Test Designation	Railing Design	Test Conditions	Evaluation Criteria*					Evaluation of Performance
			(1)	(2)		(3)	(4)	
				Long.	Result.			
W 1	Wood Post System	4,500/44.3/20.0	Pass	Pass	Sup.	Pass	Pass	Sup.
W 2	Wood Post System	4,500/58.9/16.3	Pass	Pass	Sup.	Pass	Pass	Sup.
W 3	Wood Post System	4,500/61.9/20.0	Pass	Pass	Sup.	Pass	Pass	Sup.
W 4	Wood Post System	2,250/63.0/18.7	Pass	Pass	Sup.	Pass	Pass	Sup.
W 5	Wood Post System	2,250/44.3/20.0	Pass	Pass	Sup.	Pass	Pass	Sup.
S 3	Steel Post System	4,500/61.7/16.6	Pass	Pass	Sup.	Pass	Pass	Sup.
S 4	Steel Post System	2,250/58.6/16.0	Pass	Pass	Sup.	Pass	Pass	Sup.
S 6	Steel Post System	2,250/60.0/16.0	Pass	Pass	Sup.	Pass	Pass	Sup.

*Refers to Table 13

1 lb = 0.454 kg

1 mph = 1.609 kph

V. SUMMARY AND CONCLUSIONS

A total of 30 full-scale vehicle crash tests were conducted on five in-service railing designs and an instrumented wall. The in-service railing designs were selected to be representative of those employed throughout the nation. It was originally intended that only railings meeting the AASHTO Specifications (17) be considered for testing and evaluation. However, upon closer study, it was discovered that the geometry of many railing systems was not within the limits that the specifications stated it "should" be. The "should" statement was not being interpreted as an absolute requirement and these railing systems were being installed under the specifications. After this discovery, FHWA took immediate corrective action (22).

The five in-service railing systems selected for testing and evaluation were:

- Colorado Type 5 (steel)
- Texas T101 (steel)
- New Hampshire Two-Bar Aluminum
- North Carolina One-Bar Aluminum on Concrete Parapet
- Indiana Type 5A Aluminum and Modified Type 5A

The series of tests conducted on the Colorado Type 5 railing demonstrated that it has undesirable geometrics in that the 0.38 m (15 in.) open space between the curb and metal railing extends from 0.23 to 0.61 m (9 in. to 24 in.) above the roadway surface. This open space allowed excessive penetration of the automobile bumper and front wheel with subsequent snagging on posts. A 9,080 kg (20,000 lb) school bus test demonstrated that the railing had adequate strength to contain the vehicle but insufficient height to prevent rollover.

In the series of automobile tests on the Texas T101 railing, very clean, smooth redirections were obtained. However, tests with a Honda Civic were not conducted on this system. The strength of this railing was adequate to contain a 9,080 kg (20,000 lb) school bus.

Automobile tests conducted on the New Hampshire Two-Bar Aluminum railing showed that this design has undesirable geometrics. The presence of the protruding curb coupled with the vertical location of the open space

caused severe snagging and excessive damage to the automobile front wheel.

Geometric inadequacies and a structural deficiency in the baseplate of the Indiana Type 5A railing were discovered in the series of automobile tests conducted on it. The weak failure mode of the baseplate also existed in the New Hampshire design but its influence was overshadowed by other deficiencies.

Results of testing and evaluation performed on the in-service railings and other recent crash testing have provided information useful in designing bridge railing systems that would be expected to perform in an acceptable manner. This information has been formulated into design guidelines and presented in Chapter III. These guidelines address vehicles ranging in size from an 817 kg (1,800 lb) Honda Civic to a 14,528 kg (32,000-lb) intercity bus, and set forth desirable geometric as well as strength guidelines.

If these guidelines are followed, bridge railings with improved crashworthiness will be achieved. However, the data on which they are based is limited and there is no guarantee that all new railing designs will be completely adequate in all features. It is recommended that these guidelines be used in the development of improved bridge railing designs but that final acceptance of a design be based on performance demonstrated through full-scale crash tests.

Key features of the recommended design guidelines are:

- (1) When considering the height of railing necessary to prevent rollover of a vehicle, the effective height rather than the total geometric height should be considered. The effective height is defined as the distance from the pavement surface to the centroid of the resisting force provided by the railing. Recommended effective railing heights are given in Table 7.
- (2) Curbs should not be employed unless they are set behind the traffic face of the railing. A curb, if used, should be ignored when determining the vertical open space immediately above the deck.
- (3) Vertical open space between the deck and lower extremity of

the bottom rail element should be in accordance with Figure 84.

- (4) It is highly desirable to provide blockage (with one or more rail elements) of the vertical distance from 0.23 or 0.25 m (9 or 10 in.) above the deck to 0.56 m (22 in.) above the deck in order to accommodate the range of automobile sizes.
- (5) Results from a series of tests on an instrumented wall combined with selected results from tests on in-service railings and analytical work have resulted in recommended design loads and design procedures for nominally rigid railing systems to accommodate vehicles ranging from automobiles to intercity buses. Recommended design loads and distribution patterns are given in Table 12. These design loads should be used with ultimate strength failure mechanism procedures, yield strength of the materials, and a factor of safety of 1.0 when the material and structural elements being used have adequate ductility and an ultimate strength much greater than yield strength.

If these recommendations are followed along with good engineering practices in designing bridge railing systems, there is a reasonably good probability that acceptable performance will be achieved. The ability to accomplish suitable performance for automobiles through design alone is not perfect but is more mature than it is for heavier vehicles. Impact performance of railing systems for heavier vehicles is simply a much newer subject area. For this reason and because of some accident experiences, performance standards requiring full-scale testing of bridge railing systems have been developed. It is recommended that all bridge railing systems be required to meet these standards.

Several reasons for recommending that performance testing be required were disclosed in testing and evaluation reported herein. The most prevalent deficiency discovered in railing systems was inadequate geometry. Dynamic interaction between a railing and various vehicles with wide ranging geometrics and suspension systems is a complex phenomenon and is very difficult to quantify without excessive conservatism. Full-scale

testing is an attractive means of circumventing assumptions and conservatism concerning geometrics in prescription specifications.

Performance standards based on full-scale crash tests have been developed. They are presented in Chapter IV (Table 13). The acceptable limits for automobile longitudinal accelerations were derived from tests on a flat-faced concrete wall and are used to define railings with unacceptable snagging characteristics. Acceptable limits and superior limits for the resultant accelerations imposed on automobiles were based on selected probability of injury levels. Rigid railings with good geometric characteristics will generally meet the acceptable acceleration limitations. Advanced railings with controlled deformation or yielding characteristics will be required to meet the superior acceleration limitations.

Use of the recommended Design Guidelines and Performance Standards will result in structurally sound and functionally safer bridge railings.

REFERENCES

1. Michie, Jarvis D., Bronstad, Maurice E., Kimball, C. E., Jr., and Miles, Edwin O., "Upgrading Safety Performance in Retrofitting Traffic Railing Systems", Final Report on Contract DOT-FH-11-8100, Southwest Research Institute, San Antonio, TX, September 1976.
2. Bronstad, M. E. and Kimball, C. E., Jr., "Multiple Service Level Bridge Railings - Performance and Design Criteria", Phase II Report on NCHRP Project 22-2 (2), Southwest Research Institute, San Antonio, TX, April 1979.
3. "Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances", Transportation Research Circular No. 191, Transportation Research Board, National Academy of Sciences, Washington, D. C., February 1978.
4. Bronstad, M. E., Calcote, L. R., and Kimball, C. E., Jr., "Concrete Median Barrier Research", Volume 2, Research Report, Southwest Research Institute, San Antonio, Texas, March 1976.
5. Hirsch, T. J., Panak, John J., and Buth, C. E., "Tubular W-Beam Bridge Rail", Research Report 230-1, Texas Transportation Institute, Texas A&M University, October 1978.
6. Hirsch, R. J., "Analytical Evaluation of Texas Bridge Rails to Contain Buses and Trucks", Research Report 230-2, Texas Transportation Institute, Texas A&M University, August 1978.
7. Hirsch, T. J., "Bridge Rail to Restrain and Redirect Buses", Research Report 230-3, Texas Transportation Institute, Texas A&M University, October 1980.
8. Hirsch, T. J., "Bridge Rail to Restrain and Redirect 80,000 lb Trucks", Research Report 230-4F, Texas Transportation Institute, Texas A&M University, November 1981.
9. Michie, Jarvis D., "Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances", NCHRP Report 230, 1981.
10. Viner, John G., "Vehicle Downsizing and Roadside Safety Hardware", American Association for Automotive Medicine, Proceedings, Louisville, KY, October 3-6, 1979.
11. Personal communication from John G. Viner, Office of Research, FHWA.
12. Personal communication from James H. Hatton, Office of Engineering, FHWA.

13. Olson, Robert M., Post, Edward R., and McFarland, William F., "Tentative Service Requirements For Bridge Rail Systems", National Cooperative Highway Research Program Report 86, Texas A&M University 1970.
14. Olson, R. E., Weaver, G. D., Ross, H. E., Jr., and Post, E. R., "Effect of Curb Geometry and Location On Vehicle Behavior", National Cooperative Highway Research Program Report 150, Texas Transportation Institute, Texas A&M Research Foundation, 1974.
15. Buth, C. E., Hayes, G. G., Arnold, Althea, and Dolf, Tim, "Aluminum Bridge Rail Systems", Research Reports 4182-1, 2, 3, Texas Transportation Institute, Texas A&M Research Foundation, February 1980, March 1980, and February 1981.
16. Campise, Wanda L., and Buth, C. E., "Crash Worthiness of Aluminum Tru-Beam Bridge Railing", Final Report on Contract DTFH-81-P-30117, Texas Transportation Institute, September 1982.
17. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges", Washington, D.C., 1976.
18. American Association of State Highway and Transportation Officials, "Standard Specifications for Highway Bridges", Washington, D.C., 1979.
19. National Transportation Safety Board, "Safety Effectiveness Evaluation of Traffic Barrier Systems", Report NTSB-SEE-80-5, Washington, D. C., September 30, 1980.
20. Bronstad, M. E., and Kimball, C. E., Jr., "Multiple Service Level Bridge Railings - Performance and Design Criteria", NCHRP Report No. 22-2(2), (Final Report, SwRI Project 03-4634), April 1979.
21. Kimball, E. E., Bronstad, M. E., Michie, J. D., Wentworth, J. A., and Viner, J. R., "Development of a Collapsing Ring Bridge Railing System", Report No. FHWA-D-76-39, (Final Report, SwRI Project 03-3619-1), January 1976.
22. Memorandum from W. J. Wilkes, Director of Engineering to Regional Federal Highway Administrators, June 7, 1977.
23. Society of Automotive Engineers, "SAE J885a, Human Tolerance to Impact Conditions as Related to Motor Vehicle Design", SAE Handbook, April 1980.
24. Martin, D. E., "Highway/Vehicle Compatibility-The Continuing Challenge".
25. "A Guide to Standardized Highway Barrier Rail Hardware", ARBA Technical Bulletin 268-A, American Road Builders Association, Washington, D. C., July 1973.

26. Blodgett, O. W., Design of Welded Structures, The James F. Lincoln Arc Welding Foundation, 1972.
27. Bronstad, M. E. and Michie, J. D., "Recommended Procedures for Vehicle Crash Testing of Highway Appurtenances," NCHRP Report 153, 1974.
28. Michalski, C. S., "Model Vehicle Damage Scale: A Performance Test," Traffic Safety, Volume 12, Number 2, June 1968.
29. "Specifications for Aluminum Bridge and Other Highway Structures", The Aluminum Association, Washington, D. C., 1976.
30. "Manual of Steel Construction", Eighth Edition, American Institute of Steel Construction, Inc., Chicago, Ill., 1980.
31. Arnold, A., Campise, W., and Buth, E., "Thrie-Beam Weak-Post Guardrail System", Texas Transportation Institute, Texas A&M University, June 1982.
32. Ivey, Don L., Robertson, Richard, and Buth, C. E., "Test and Evaluation of W-Beam and Thrie-Beam Guardrails", Texas Transportation Institute, Texas A&M University, March 1982.
33. Ivey, D. L., Ross, H. E., Hirsch, T. J., Buth, C. E., and Olson, R. M., "Portable Concrete Median Barriers: Structural Design and Dynamic Performance", Transportation Research Record 769, National Academy of Sciences, Washington, D. C., 1980.
34. Kimball, C., E., Jr., Bronstad, M. E., and Michie, J. D., "Heavy-Vehicle Tests of Tubular Thrie-Beam Retrofit", Transportation Research Record 796, Transportation Research Board, Washington, D. C., 1981.

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect

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0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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